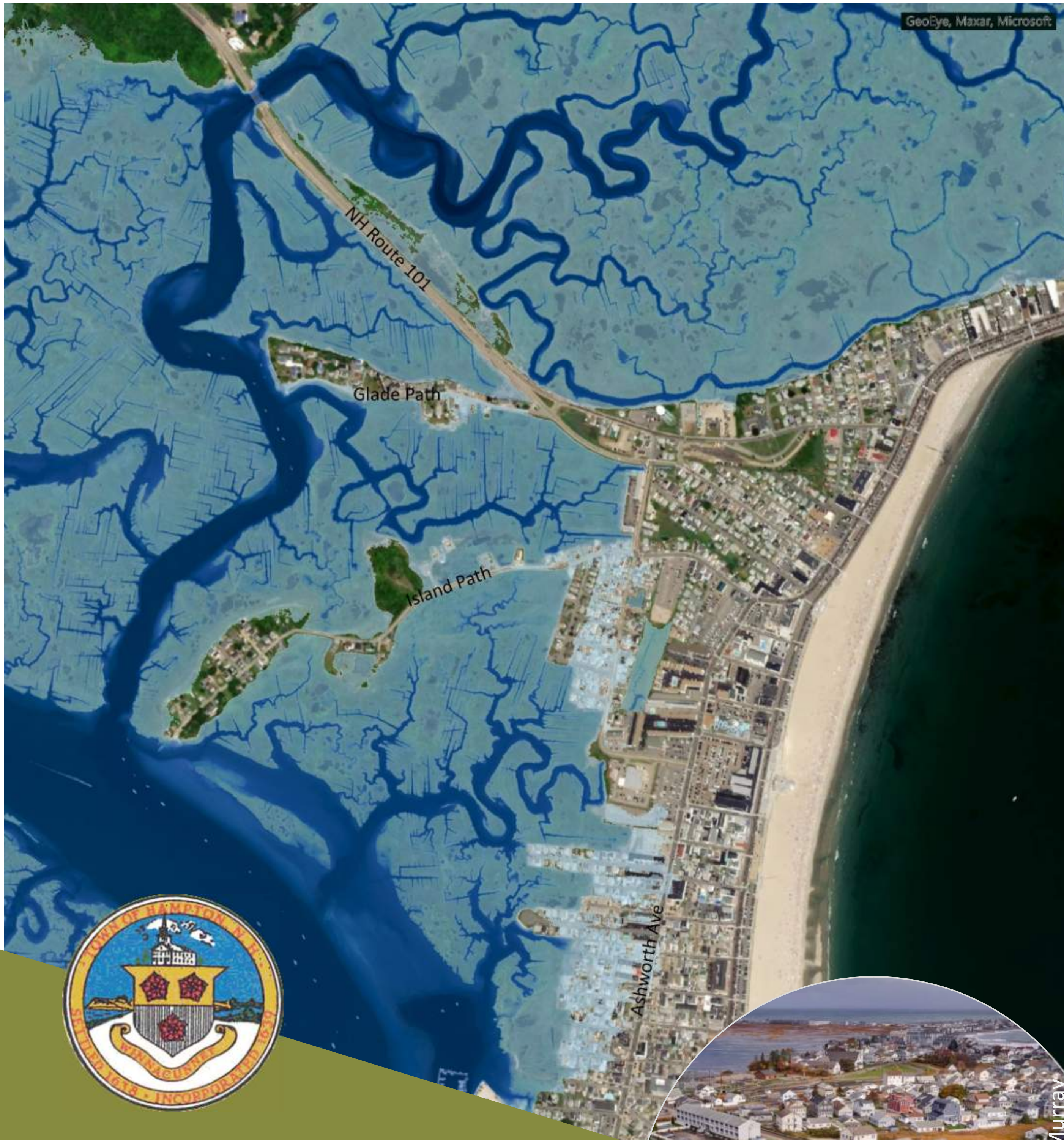
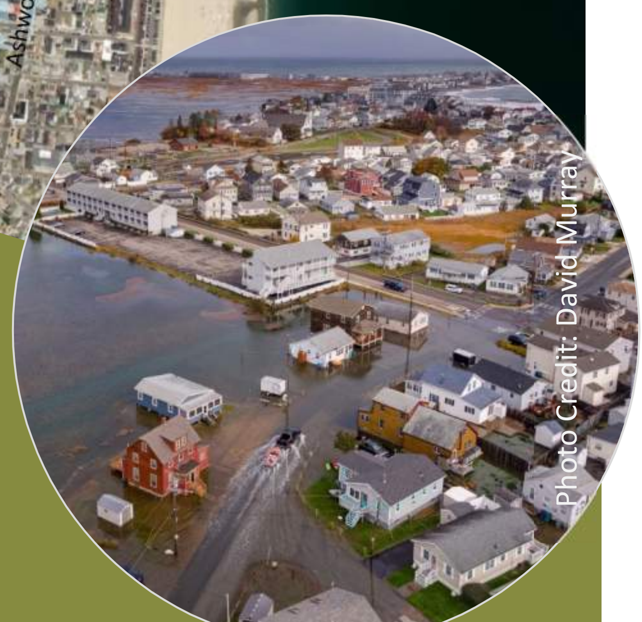


Hampton Harbor Flooding Evaluation

March 31, 2021

Prepared by:

Hoyle, Tanner & Associates, Inc.



Hoyle, Tanner & Associates, Inc.
150 Dow Street
Manchester, New Hampshire 03101
603-669-5555
www.hoyletanner.com

March 31, 2021

VIA EMAIL

Ms. Jennifer Hale, PE
Deputy Director
Hampton Department of Public Works
100 Winnacunnet Road
Hampton, NH 03842

Re: Hampton Harbor Hydraulic Assessment
Hampton, New Hampshire

Dear Jennifer:

As requested, Hoyle, Tanner & Associates, Inc. (Hoyle, Tanner) completed a review of the recurring Harbor flooding events, looking for opportunities to minimize or eliminate disruption to residents and municipal operations during flood events. Mitigation measures were evaluated in two main categories generalized as either "move the water" or "move the facilities".

As suspected, the regulatory agencies likely will not be favorable to support large-scale efforts to 'move the water' away from the residents and/or your facilities. Upon closer examination, the magnitude of costs to accomplish many of these options were deemed to be prohibitive. However, long-term planning may be prudent to relocate affected infrastructure, a viable option.

Please find attached our report summarizing our findings and suggested subsequent steps. If you have any questions, please feel free to contact us at any time.

Respectfully,

Matt Low   Heidi Marshall

Matthew Low, PE
Senior Vice President

Heidi Marshall, PE
Senior Engineer

Enclosures

Hampton Harbor Flooding Evaluation

Town of Hampton, New Hampshire

Prepared for:

Town of Hampton, New Hampshire

By:

Hoyle, Tanner & Associates, Inc.

March 31, 2021

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EXECUTIVE SUMMARY

Hampton's residents and guests have indicated that they have been subjected to more frequent and impactful flooding events throughout recent history. Hampton is searching for opportunities to plan for and to minimize disruptions from flooding. The purpose of this Study, which focuses on the Harbor and adjacent marsh area, is to look at opportunities for Hampton to modify current practices and to identify improvements that can be planned or constructed to adjust water's negative impacts on property, infrastructure and quality of life. This report was developed in a cooperative effort with Milone & MacBroom, Inc., now part of SLR International Corporation.

It is understood that no one community in New Hampshire is able to solve the cause of sea level rise; however, negative community impacts may be reduced by teaming with others having common concerns. Prior to beginning the Study, both members of the public and Town representatives had ideas as to how to solve the water encroachment problem. The Study results indicate that although most solutions to stop the water from entering the buildings are exceptionally effective, these solutions may create new impacts and challenges.

One commonly used phrase associated with flooding is Avoid, Accommodate, Resist, and Relocate. Each of the major action items identified in the Study fall into one of the categories: Avoid (Zoning and Site Plan Regulation Revisions); Accommodate (Raise Buildings); Resist (Temporary/Permanent Flood Walls and Raise Roads); and Relocate (Managed Retreat).

No one alternative is strongly recommended at this time, however; viable short-term solutions include the use of temporary barriers and support of continued revisions to the Zoning Ordinances/Site Plan Regulations. The most feasible longer-term solutions require elevating key roads and voluntary and/or assisted retreat. Managed retreat of affected structures out of the flood-prone areas should be seriously considered and prioritized looking towards a cooperative effort with FEMA.

Although to better understand feasibility, further investigation must be completed for the permanent barrier and elevated road options, including permitting, constructability, cost, and potential funding sources/partners. As alternatives are advanced, it is also critical to monitor sea level rise, new flood protection technology, and other solutions that others maybe implementing for flood control.

Alternatives commonly discussed to minimize flooding and sea level rise often include options such as identification and construction of compensatory storage, stormwater pump stations construction, and expansion of beaches (that can be used to minimize impacts of storm surges). Due to the potential for adverse environmental impacts, the volume of water that will need to be managed and the proximity of the impacted area to the valuable marshland, these alternatives were not deemed viable to further evaluate during this phase of Study.

1. INTRODUCTION

Hoyle, Tanner & Associates, Inc. (Hoyle, Tanner) was retained by the Town of Hampton, New Hampshire to assess the recurring flooding near the Harbor. This Study focuses on the back bay area including Glade Path, Brown Avenue, Island Path, Battcock Avenue, and south along Ashworth Avenue and intersecting side streets to the Harbor. This was divided into three sub-areas of interest to focus on: Glade Path, Island Path, and Ashworth Avenue. Milone & MacBroom, Inc (MMI), now part of SLR International Corporation (SLR), completed the majority of the hydrologic and hydraulic modeling efforts for the project. MMI is evaluating the flooding in the Meadow Pond area and potential flood mitigation alternatives for that area. This Study summarizes our understanding of the project, our approach, and findings from our evaluation.

1.0 PURPOSE

The purpose of this Study is to evaluate the existing and future flooding trends in the Harbor, with focus on the back bay area, and to evaluate potential flood mitigation alternatives. It is important to note that the evaluation of flood mitigation alternatives shown in this report is exploratory. Many of the preferred alternatives take place on public land such as in a roadway. The implementation of an alternative on private property will require future discussions with property owners and will only take place with landowner agreement. The taking of property is not proposed and has never been considered as part of this project.

1.1 OBJECTIVES

The primary objectives of this evaluation are for the stakeholders to gain a better understanding of the flooding in the Harbor area and for Hoyle, Tanner to present potential "big picture" strategies to mitigate flooding or the effects of flooding on the Town using strategies including:

- Modeling of existing conditions using current and future water levels, including sea-level rise and storm scenarios, and modeling of prioritized flood mitigation alternatives analysis;
- Development of a set of prioritized conceptual recommendations related to flood damage mitigation; and
- Coordinating an improved understanding of feasible options and associated costs among key Town officials and stakeholders.

1.2 PROJECT APPROACH

Hoyle, Tanner developed a multi-step approach to meet the objectives of the project. The project team, consisting of civil and environmental engineers, met with stakeholders to review the flood impact areas, flood stage levels, the Town of Hampton's response to flooding events, and potential mitigation alternatives.

Hoyle, Tanner and MMI staff reviewed publicly available flood maps, USGS maps, USGS stream data, and available hydraulic studies for data on the harbor to gain a better understanding of the extents of past flood events and the associated flows during those events.

From review of available information, potential "large-scale" flood mitigation strategies were identified and conceptual level "order of magnitude" opinions of cost for implementation of these strategies were developed. The relative "order of magnitude" costs of the mitigation strategies can be weighed by the Town against the annual cost of losses due to flooding in the area.

It is noted that, due to the proximity of the back bay area to the mouth of the Harbor, some mitigation strategies that may be effective in the further reaches of the marsh area, such as phragmite removal and creation of additional flood storage area via either dredging or removal of structures and fill from the floodplain, have minimal impact on water surface elevations within the Study Area and therefore would not be effective. The NH Route 101 corridor acts as a barrier to divide the marsh between the back bay area to the south and the marshes to the northwest. During flooding events that do not inundate NH Route 101, all flow entering the northern marsh is routed through the hydraulic opening of the NH Route 101 bridge, restricting flow and greatly contributing to the tidal lag observed in areas north. There is minimal tide lag in the Harbor itself (south of NH Route 101) where water surface elevations are driven more directly by the ocean. This decoupling makes the water surface elevations in these northern marsh areas more sensitive to other flood mitigation strategies, especially in the Meadow Pond area where MMI's efforts focused because of a second constriction at the Winnacunnet Road (NH Route 101E) bridge.

With the potential mitigation strategies defined, the team contacted representatives at the NH Department of Environmental Services (DES) and the US Army Corps of Engineers (USACE) to discuss possible large-scale mitigation strategies. Regulatory personnel were asked to provide their opinions on the feasibility of permitting these large-scale mitigation strategies, as well as the expected timeline for approvals. Funding for feasible mitigation measures was also discussed with DES and USACE staff.

The findings from this evaluation are summarized below and suggested "next steps" for The Town of Hampton to take in implementing flood mitigation strategies are provided.

1.3 ACKNOWLEDGEMENTS

Background information for this report has been secured from a diverse group of individuals including those directly affected by flooding, those observing from the outside, and individuals dedicated to managing adverse impacts to the New Hampshire environment.

Development of the report has been a collaborative effort between the Town of Hampton Department of Public Works, Hampton Harbor area residents, Hoyle, Tanner, MMI, Doucet Survey, Woods Hole Group, the University of New Hampshire Institute for the Study of Earth, Oceans and Space, and the Coastal Program of the New Hampshire Department of Environmental Services.

First, thank you to the residents of the Harbor Study area for providing photos, details, and accounts of the various flooding situations and how the flooding impacts have evolved throughout this century. Additionally, this Study would not have been completed without the creativity demonstrated by Doucet Survey and the University of New Hampshire with respect to data collection. Thank you to MMI and the Woods Hole Group for providing data and developing the model used for the evaluations.

2. BACKGROUND

Please note, MMI developed the model with additional data from the Woods Hole Group. Since the MMI modelling is the foundation for the evaluations, and in an effort not to reword or change the meaning of the elements discussed by MMI, the section headings containing "(MMI, 2021)" throughout this Study are taken directly from the Hampton Flood Mitigation Analysis Final Engineering Report prepared by MMI (2021); certain portions of the sections unique to the Meadow Pond area and not applicable to the Harbor were removed, and supplemental information contained within brackets was added for this Study.

2.0 BACKGROUND (MMI, 2021)

Chronic tidal flooding and storm-based overland flooding are common in Hampton. The frequency of flooding is increasing, especially in the [Study area]. A hydraulic model was set up for most of the Hampton-Seabrook estuary (Figure 1) to understand the flow from the tide and freshwater inputs and evaluate flood mitigation alternatives.

As part of Phase I of the project, preferred flood mitigation alternatives will be recommended by MMI for the Meadow Pond Area and Hoyle Tanner Associates [Hoyle, Tanner] for the Harbor Area. This report documents the results of the hydraulic modeling and the flood mitigation alternatives analysis for the [Harbor] Area. Please see the MMI report for more information on the [Meadow Pond] Area.

The Town of Hampton has received funding from the National Fish and Wildlife foundation (NFWF) for phase 2 of the project to advance the design of the preferred alternatives identified here that provide resiliency to the salt marsh in the form of flood mitigation and habitat protection.

Figure 1 Project Location

2.1 LAND USE (MMI, 2021)

The topography of the area is flat and dominated by salt marsh with development around the edges (Figure 2). Route 101, Winnacunnet Road, and High Street travel east-west across the marsh. These road embankments restrict natural movement of both tidal and freshwater flows and are one of the causes of flooding.

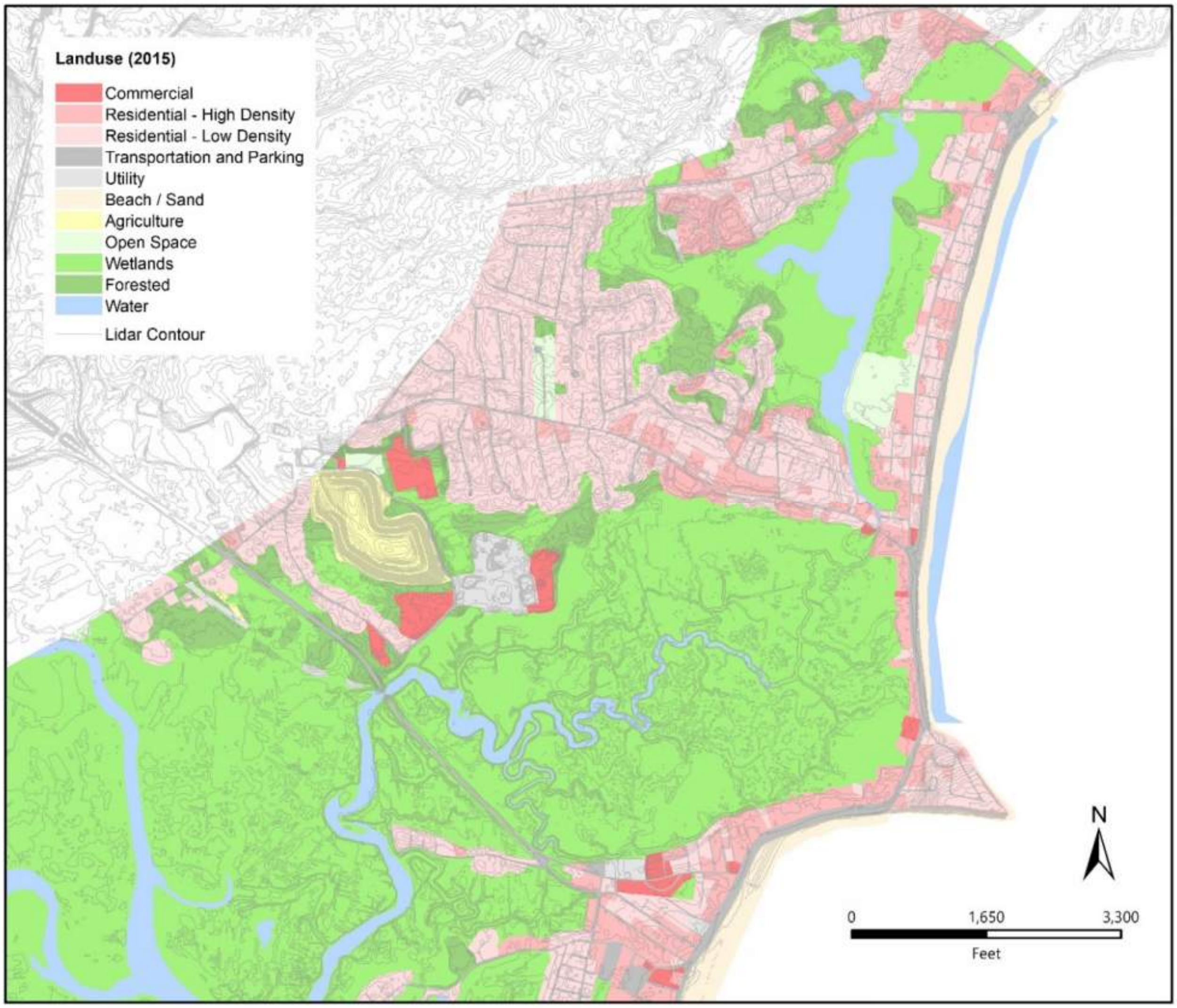


Figure 2 Land Use

2.2 FLOODING (MMI, 2021)

The Hampton Hazard Mitigation Plan (Hampton, 2016) lists coastal and river flooding as one of the top hazards in the town that severely impacts property and business, occurs frequently, and has severe risk. Flood issues are present along the coast and floodplains (Appendix A). Again, [MMI's] Study focuses on flood issues associated with the Meadow Pond area [and Hoyle, Tanner's Study focuses on flood issues associated with the Harbor area]. Flooding in [these areas] originates from increasing high tides (Table 1 and Table 2) as well as overland stormwater runoff due to inadequate drainage due to flat land, nowhere for the water to go during high tide, and deteriorated drainage infrastructure. The impacts of sea-level rise are being felt in project area based on increased chronic tidal flooding extending above the nuisance flood level of [4.8] feet NAVD 88, or 10 feet mean lower low water (MLLW).

Table 1
Number of High Tides Over 10 Feet MLLW per Year (Using High and Low Tides Dataset)
(Analysis by the NHDES Coastal Program, 2020)

Year	# of Days with Data	# of High Tides >10ft	# of Days with High Tide >10ft	Max Height (ft)	% of Days with Data & High Tide >10ft
2013	309	144	113	11.78	36.57%
2014	365	182	138	12.34	37.81%
2015	365	145	117	12.43	32.05%
2016	93	37	27	11.70	29.03%
2017	365	161	119	12.15	32.60%
2018	365	199	143	13.24	39.18%
2019	244	137	104	12.00	42.62%
2020	287	134	108	11.80	37.63%

Table 2
Number of High Tides Over 11 Feet MLLW per Year (Using High and Low Tides Dataset)
(Analysis by the NHDES Coastal Program, 2020)

Year	# of Days with Data	# of High Tides >11ft	# of Days with High Tide >11ft	Max Height (ft)	% of Days with Data & High Tide >11ft
2013	309	24	23	11.78	7.44%
2014	365	26	24	12.34	6.58%
2015	365	14	12	12.43	3.29%
2016	93	10	9	11.70	9.68%
2017	365	31	30	12.15	8.22%
2018	365	48	43	13.24	11.78%
2019	244	24	22	12.00	9.02%
2020	287	26	23	11.80	8.01%

The residents of [the Study area] are regularly inundated with water ponding on streets and intersections (Figure 3). Ponding and flooding…are common when rain occurs during high tide. Storm drainage pipe outlets are submerged and tidal water flows into low-lying areas. The ponded water is a hazard for motorists and pedestrians due to narrower travel lanes. The ponded water can also freeze during the winter and create more risks (Figure 4). Ponded water has also caused property damage to some of the homes in lower lying areas including electrical issues, damaged fencing, and unusable driveways.

Figure 3 Flooding on Perkins Ave

Figure 4 Iced Over Ponding on Gentian Road (Bergin, 2018)

…Limited conventional gravity drainage solutions are possible due to the flat topography, low elevation, and rising tides, [which also] lead to elevated groundwater levels…In summary, inadequate road grading, saturated soils, and lack of a drainage system are leading to water ponding in the road during rainy periods and high tides…

2.3 HARBOR FLOODING

The Town of Hampton has endured a long history with portions of the beach area flooding during high tide events, particularly in spring when high tide is paired with a coastal storm. The beach area is surrounded by marshland on one side (the Harbor area) and beach on the other side. When the tide rises, the flooding of the Harbor has inundated the Town of Hampton's buildings, roadways, and parking areas causing significant property damage with each flood event (Figure 5). Table 1 details the extent of flooding based on the water level at the tide gauge located near the Harbor. Gauge data from the last eight years indicate 30% to 40% of days for any given year have high tides that reach the action stage, the level at which significant water activity is possible (see full definition included in Table 3). This data also illustrates that six to ten percent of days per year experience flooding above the flood stage.

Figure 5 Historic documentation of disruptive flooding (Town of Hampton)

Table 3
Hampton Harbor Flood Stage Impact Table (NGVD 88*, feet)

Elevation	Impact
4.81	Action Stage**
5.81	Onset of flooding along low lying streets in the Hampton Harbor area
6.81	Roads flooded up to a foot deep in the Harbor area
8.11	Cars flooded with low lying Harbor area roads in Hampton covered by up to three feet of water

*Add 5.19' to convert to MLLW

** "Action Stage - an established gage height which when reached by a rising stream, lake, or reservoir represents the level where action is taken in preparation for possible significant hydrologic activity" (NWS Manual 10-950, 2019)

Source: Unless noted otherwise, all information contained in this table is directly taken from NWS Advanced Hydrologic Prediction Service (2020)

2.4 SEA-LEVEL RISE (MMI, 2021)

Sea level has increased 0.3 feet in the last 20 years at the Portland, Maine National Oceanic and Atmospheric Administration (NOAA) tide gauge. A range of predictions exist that indicate sea level could rise between 0.6 feet and 1.7 feet in the next 30 years (Mellor, 2020) (Figure 6). The New Hampshire Coastal Flood Risk Science and Technical Advisory Panel (STAP), a DES program, has developed probabilistic relative sea level rise (RSLR) projections for the New Hampshire coast. The projections show that sea level is likely to rise between 0.5 and 1.3 feet by 2050, and between 1.0 to 2.9 feet by 2100. There is a 5% chance that RSLR will exceed 1.6 feet by 2050 and 3.8 feet by 2100. [State agencies recognize climate change considerations in design, including sea-level rise, and are planning for it. For example, NHDOT is accounting for a sea-level rise of 3.9’ by 2100 for the design of the NH Route 1A (Ocean Boulevard) bridge over the Hampton Harbor Inlet (Reczek, 2021).]

Coastal flood risk modeling results for Massachusetts and southern New Hampshire show that the typical spring high tide elevation may increase 4 to 5 feet by 2070 (Bosma et al., 2015) (Figure 7). These climate-based predictions of sea-level rise originally developed by NOAA have been used in the hydraulic modeling to evaluate the performance of alternatives in the future.

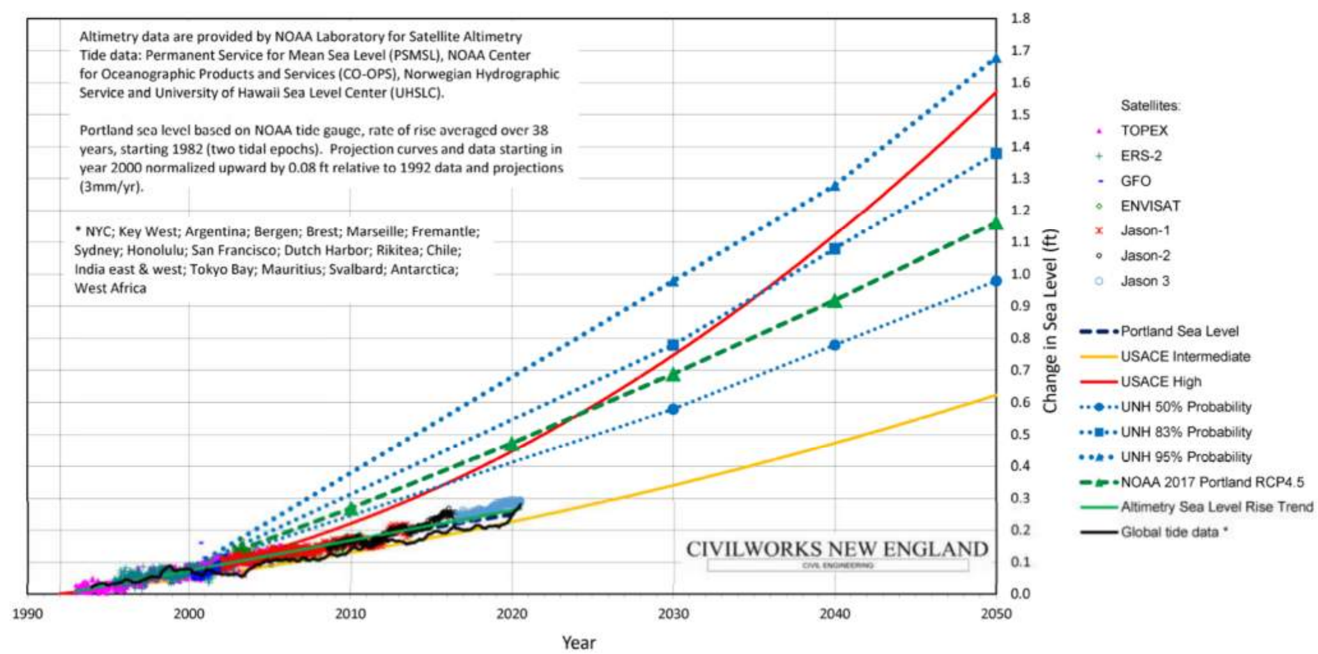


Figure 6 Sea-Level Rise Predictions 1990 to 2050

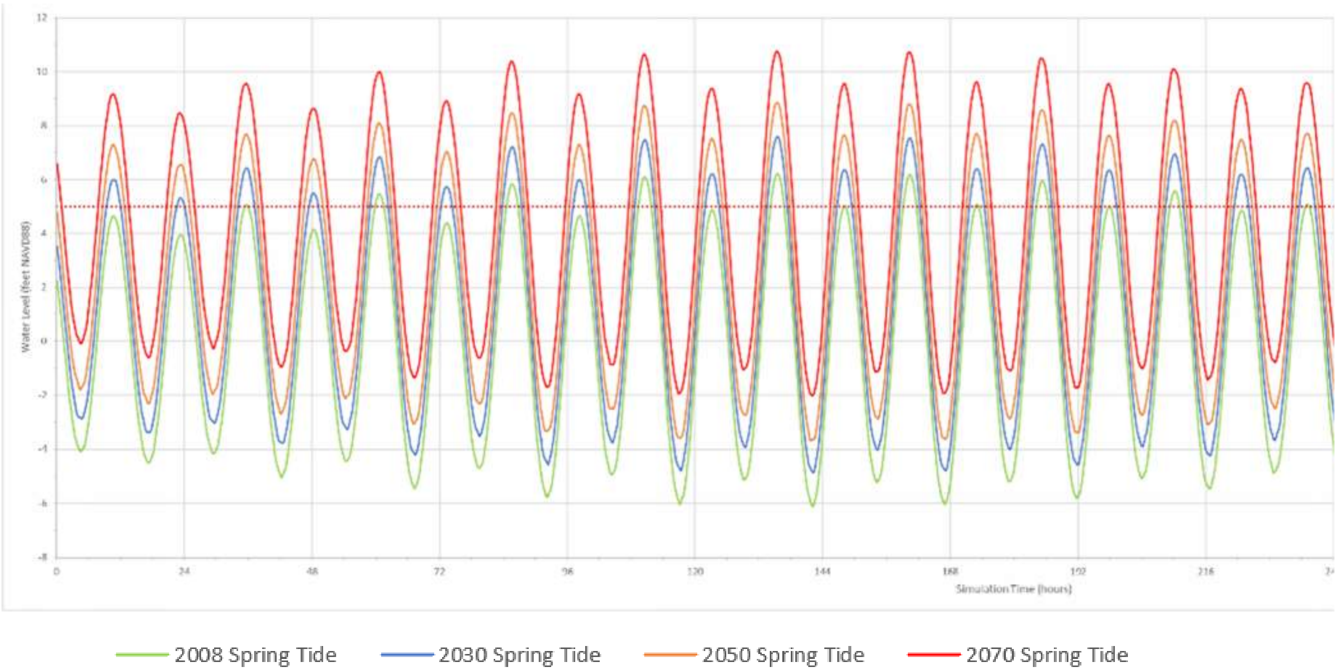


Figure 7 Summary of Predicted Spring Tide Water Levels in Hampton Harbor 2008 to 2070

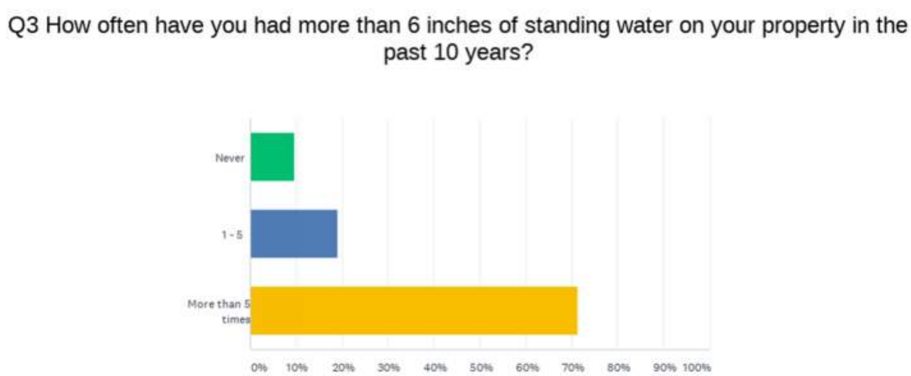
3. DATA COLLECTION

3.0 DATA COLLECTION

Prior to initiation of the Study, a public meeting was held to acquire an understanding of flooding aspects. As part of this outreach event, a questionnaire was prepared using a dual format (electronic and in-person paper-style).

Over 90% of the respondents to the questionnaire were members of the residential community, with just over 2% noted as businesses and approximately seven percent identified themselves as other, including joint residential and business uses.

Only 10% of the respondents indicated that they had never encountered more than 6 inches of standing water on their property in the past 10 years, while over 71% noted 6 inches of standing water over 5 times in the past 10 years. Over 42% noted more than a foot of water had been experienced on their property, and over 14% have experienced more than 3 feet of standing water.



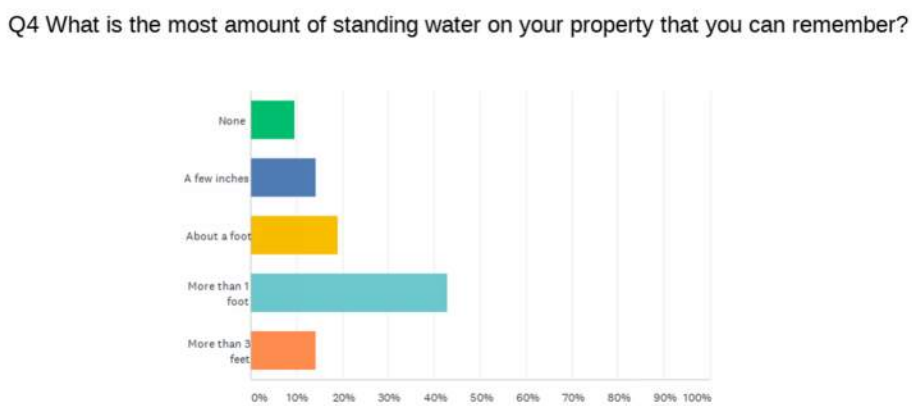


Figure 8 Questionnaire Data Results

Over 35% of respondents have experienced water in their building, likely coinciding with the 30% that indicate that water forces them to leave their building.

Over 45% indicate that flooding has become a problem for them in the past 5 years, with many citing during public outreach that they feel recent development is contributing to increased water levels.

After evaluating public input (see Appendix A), data needs were identified including water surface elevations and development of monitoring locations for various storm conditions. Over time, the data was collected to build the hydraulic model and to develop the model's test alternatives.

3.1 LiDAR (MMI, 2021)

LiDAR-derived, 2-foot contours were used to document the topography of the salt marsh and surrounding area (see the contour lines in Figure 2). The LiDAR data acquisition took place in 2014 as part of the US Geological Survey (USGS) New England CMGP Sandy LiDAR Data Acquisition and Processing Production Project. Data were acquired within 2 feet of mean low water (MLW). The resolution of the LiDAR-derived contours is 0.7 meters [2.3 feet].

3.2 SURVEY (MMI, 2021)

Topographic survey data were collected by Doucet Survey, LLC in August and September of 2019. Twenty (20) cross sections were surveyed in Meadow Pond, Eel Creek (upstream of Winnacunnet Road), and Eel Ditch (downstream of Winnacunnet Road) (Figure 9). The survey also recorded the elevation of the water level monitoring equipment (Section 3.4).

Figure 9 Surveyed Cross Sections (Red)

All survey elevations and model results are reported in the vertical datum of feet NAVD 88. Existing information used during this project has been converted to feet NAVD 88 (Table 4 and Appendix B). For example, 10 feet MLLW is equal to [4.8] feet NAVD 88. [Residents may be more familiar with the MLLW datum as most marine data/activities are referenced on MLLW.]

Table 4
Datum Conversions

Datum	Conversion to NAVD 88	Data with this Datum
NAVD 88	NAVD 88 = MLLW – [5.20]	Survey, Water Level Monitoring, Harbor Water Level, Hydraulic Model
NGVD 29	NAVD 88 = NGVD 29 - 0.784	FEMA Flood Maps
MLLW	NAVD 88 = MLLW – [5.20]	NOAA Tide Gauge, Most Common Local Elevation Reference Such as 10 for Start of Flooding
MLW	NAVD 88 = MLW – [4.83]	1946 Hampton Harbor Bridge Plans with Pier Elevations

3.3 FEMA FLOOD MAP

The Harbor area is classified as a FEMA Zone AE, which means the area has elevations determined for the base flood elevation (BFE). The BFE is the water surface elevation corresponding to an annual exceedance probability of 1% (the 100-year flood). The BFE is per the community's Flood Insurance Study (FIS) and accompanying Flood Insurance Rate Map (FIRM); it is noted that the FIS and FIRMs were Revised as of January 29, 2021. The governing BFE is the maximum elevation of that provided in FIS Table 5 – Summary of Stillwater Elevations, and the elevation provided in the FIRM. According to the FIS, "…if the elevation on the FIRM is higher than the elevation shown in [FIS Table 5], a wave height, wave runup, and/or wave setup component likely exists, in which case, the higher elevation should be used for construction and/or floodplain management purposes" (FEMA, 2021). Note that although part of the area is shown as having a BFE of 8.0' NAVD 88, the stillwater elevation of 8.36' from FIS Table 5 governs for this area. The flood hazard zone for the Harbor can be viewed using the FEMA National Flood Hazard Layer (NFHL) Viewer (2020) as shown in Figure 10, and the FIS and relevant Flood Insurance Rate Maps (FIRMs) are included in Appendix G.

Implementation of any permanent flood mitigation strategy that could affect the base flood elevations may require filing of a FEMA Conditional Letter of Map Revision (CLOMR) before alterations are made, and/or may require a FEMA Letter of Map Revision (LOMR) to be issued after the project is completed. For a permanent barrier, the LOMR would memorialize the changes to the base flood elevations due to the placement of any structure in the floodplain and loss of flood storage. However, modeling results for permanent infrastructure alternatives such as a flood barrier installed in the Harbor area indicate minimal change in water surface elevations due to loss of flood storage; see Section 5.5 for further discussion.

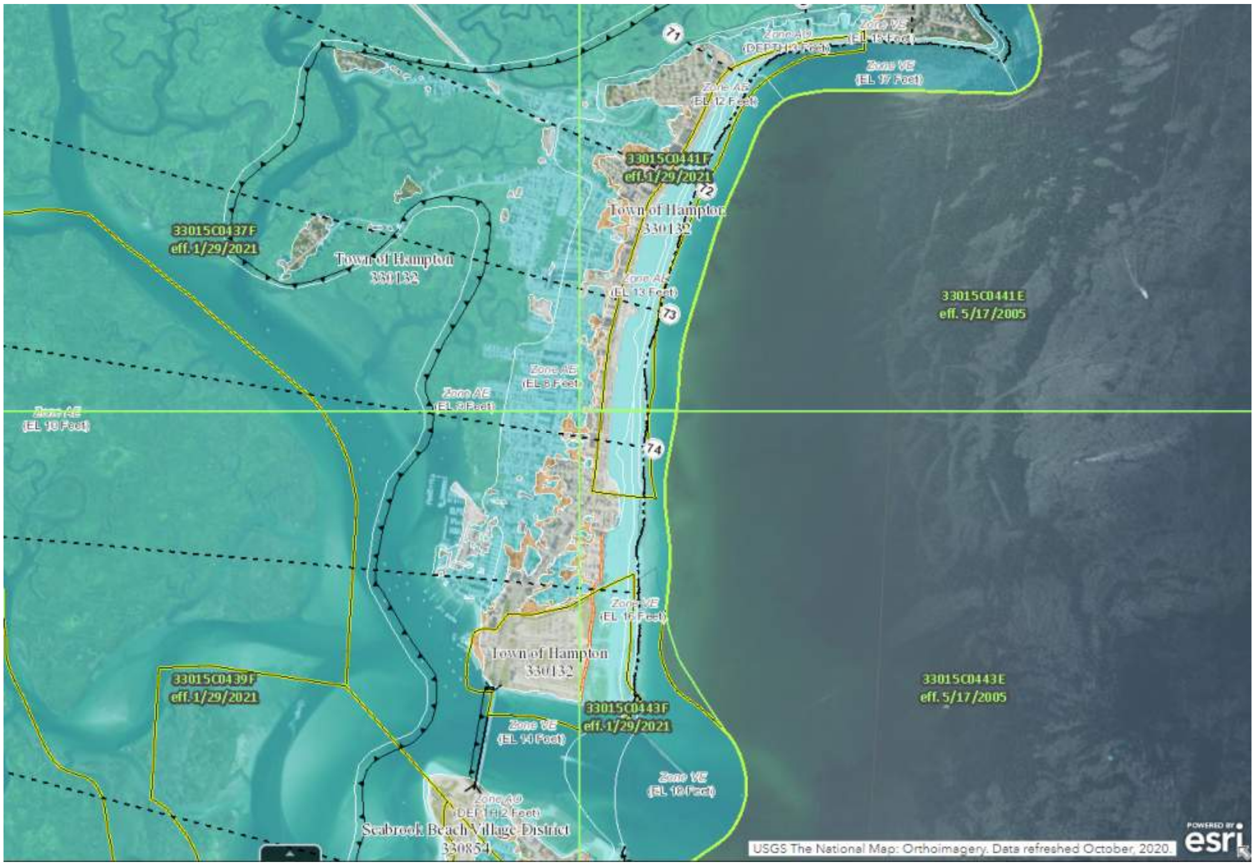


Figure 10 FEMA Flood Hazard Zone (FEMA's NFHL Viewer, 2020)

3.4 WATER LEVEL MONITORING (MMI, 2021)

The University of New Hampshire Earth Systems Research Center collected water level data at eight (8) stations across the project site (Figure 11 and Appendix C). The water level data were used to validate the hydraulic model and observe the tidal lag. The water level monitoring network remains in place for future data collection.

The water level monitoring illustrates the dampening of the tide (i.e., the flattening of the tidal amplitude) moving upstream from the Harbor to High Street (Figure 12 and Figure 13). The data also show a tidal lag (i.e., a shifting of the tide timing) moving from the Harbor to Meadow Pond. [See MMI's (2021) report for more information on tidal lag.]

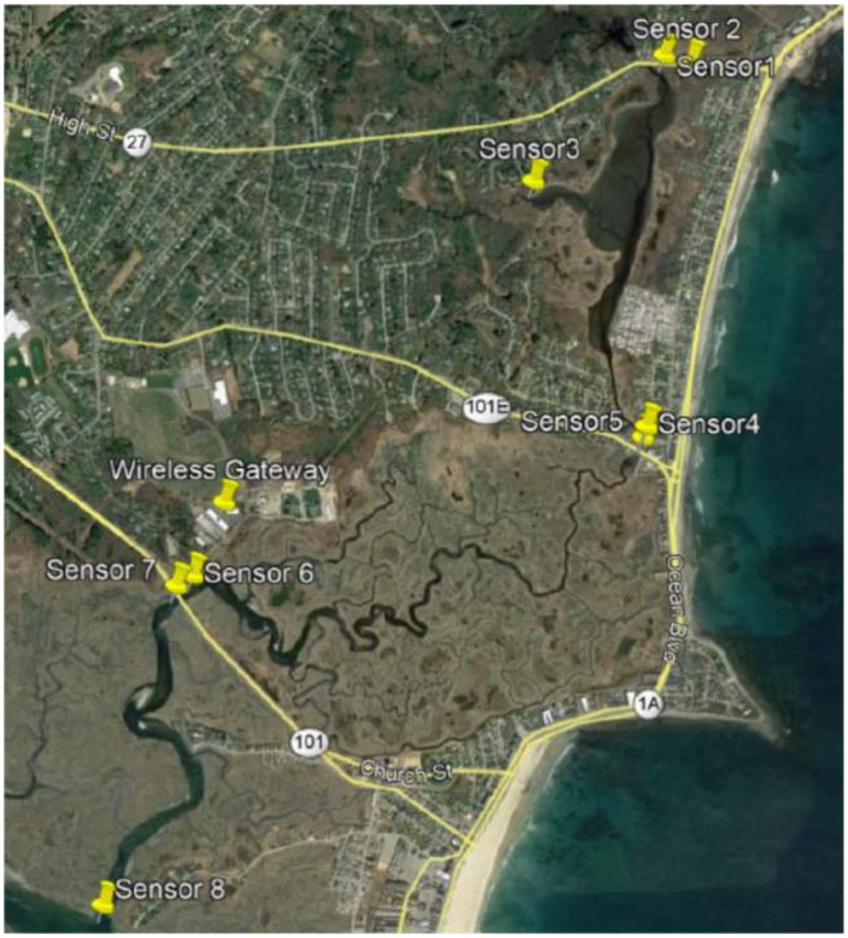


Figure 11 Water Level Monitoring Locations

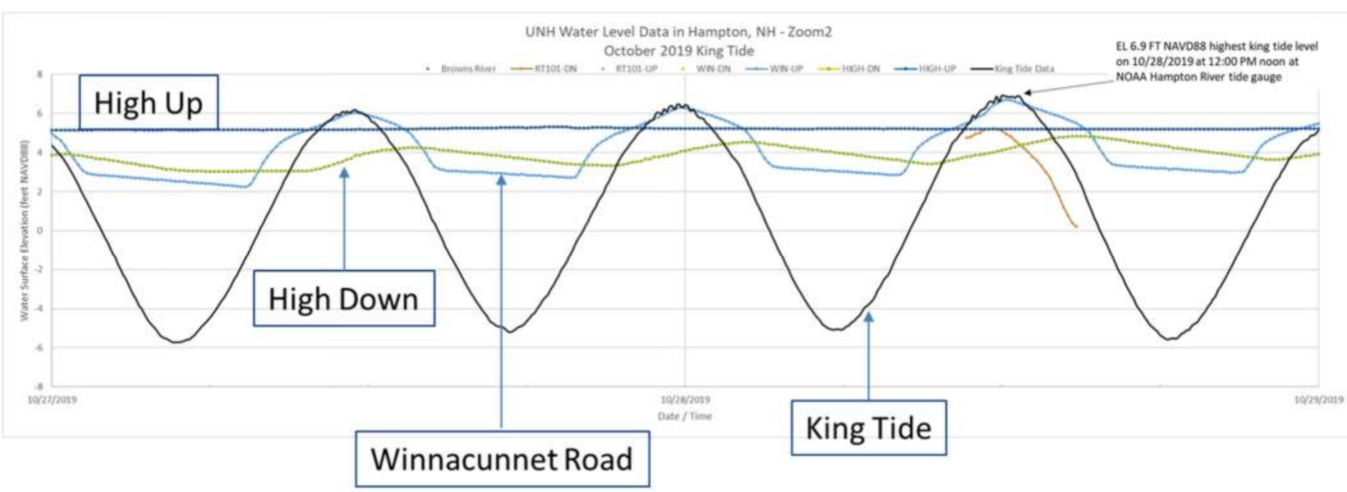


Figure 12 Sample Water Level Data for the October 2019 King Tide

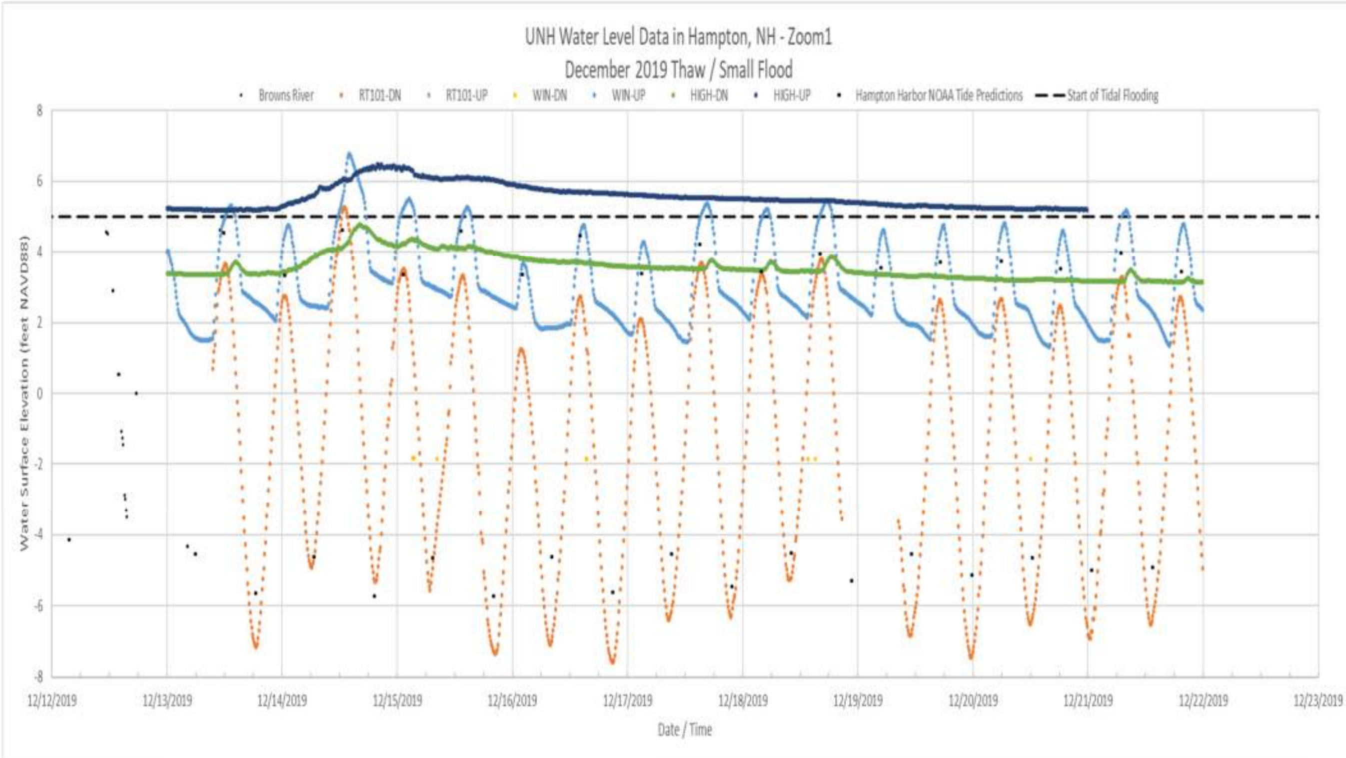


Figure 13 Sample Water Level Data for a Thaw during December 2019

3.5 *PRECIPITATION GAUGES (MMI, 2021)*

Weather stations exist that record precipitation at the Yankee Fisherman's Coop just south of the Hampton Harbor Bridge in Seabrook, NH (Station ID: KNHSEABR3) and at the Portsmouth International Airport at Pease Station located to the north and inland about 8 miles from project site (Station ID: KPSM). The rain gauge data are used to relate rain events to flood events at the project site and the data from the two stations relate weather immediately on the coast to the weather [a little] inland. For example, precipitation data during the October 2019 King Tide shows that about 1 inch of rain fell on that day at the airport while 0.25 inches fell on the coast at the project site (Figure 14). The precipitation data show that nearly 2 inches of rain fell both inland and along the coast during a December 2019 thaw where flooding took place in the region (see Figure 13). Based on NOAA (2018) precipitation frequency estimates, the December rainfall was between the 1- and 2-year rainfall event (Table 5).

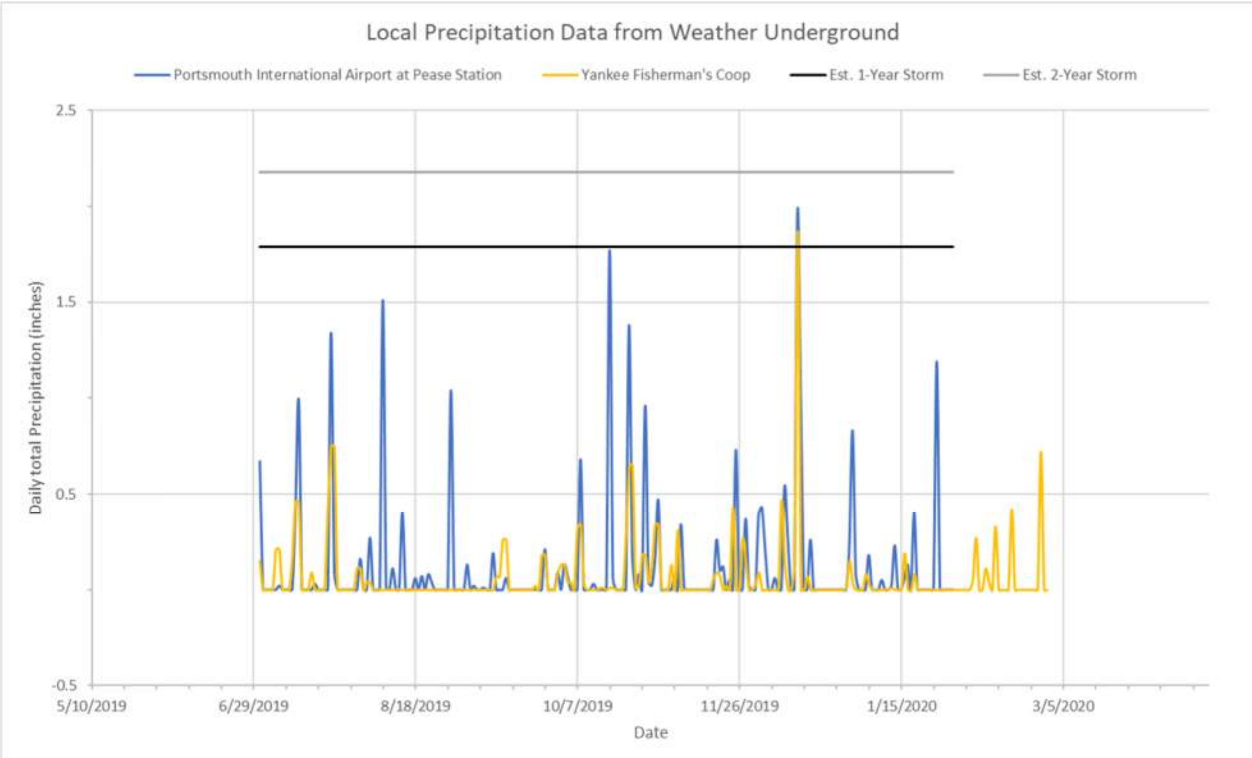


Figure 14 Sample Precipitation Data

Table 5
Rainfall Frequency Estimates (NOAA, 2018)

Average Recurrence Interval (Years)	24-Hour Duration Precipitation Estimate (inches)	6-Hour Duration Precipitation Estimate (inches)
1	2.68	1.79
2	3.33	2.18
5	4.41	2.82
10	5.29	3.35
25	6.52	4.08
50	7.42	4.62
100	8.4	5.21
200	9.64	5.92
500	11.6	7.02
1000	13.2	7.97

3.6 TIDE DATA (MMI, 2021)

Tide data were obtained from the NOAA gauge located at the Hampton River Marina (Figure 15). The tide data were used to document the October 2019 King Tide (Figure 16) and other tides. Note that water level predictions for Hampton Harbor were obtained from the Woods Hole Group Coastal Flood Risk Model as described in Section 4.2.

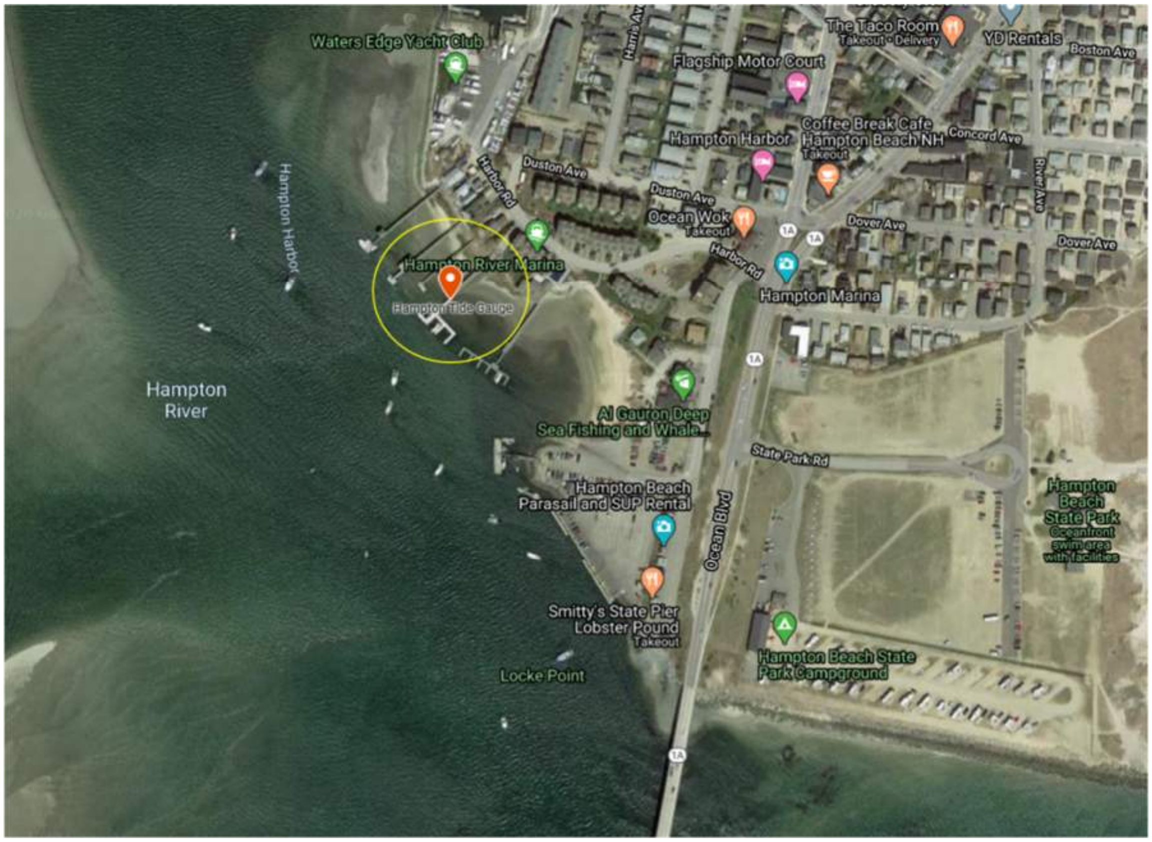


Figure 15 The Location of the Hampton River Tide Gauge

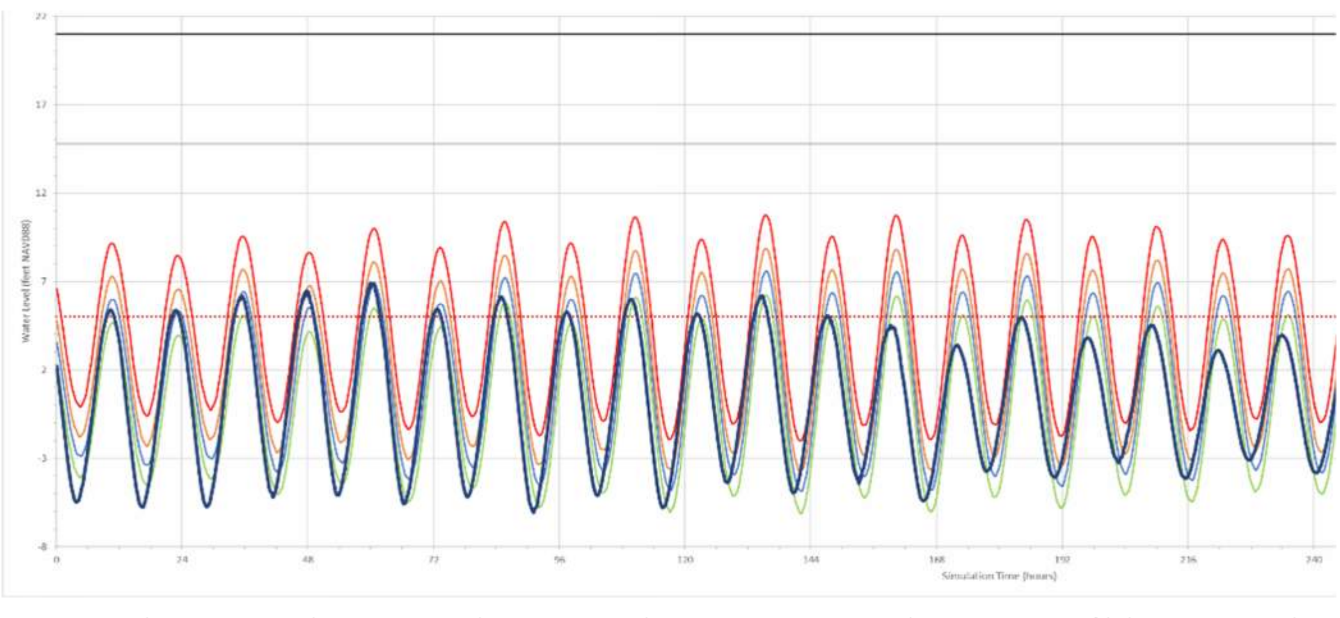


Figure 16 2019 King Tide NOAA Gauge Data Shown with the Predicted Spring Tides in 2008, 2030, 2050, and 2070 (Bosma et al., 2015), the Low and High Chords of the Hampton Harbor Bridge, and the Chronic Local Flood Level of 5 Feet NAVD 88

3.7 SEA LEVEL AFFECTING MARSHES MODEL (SLAMM) FOR NEW HAMPSHIRE (MMI, 2021)

The Sea Level Affecting Marshes Model (SLAMM) predicts a sea-level rise of 3.3 feet in 2025 and 6.6 feet in 2100 for the Hampton Seabrook Estuary. The changing sea level leads to changes to salt marsh habitat with more marsh inundation (Figure 17).

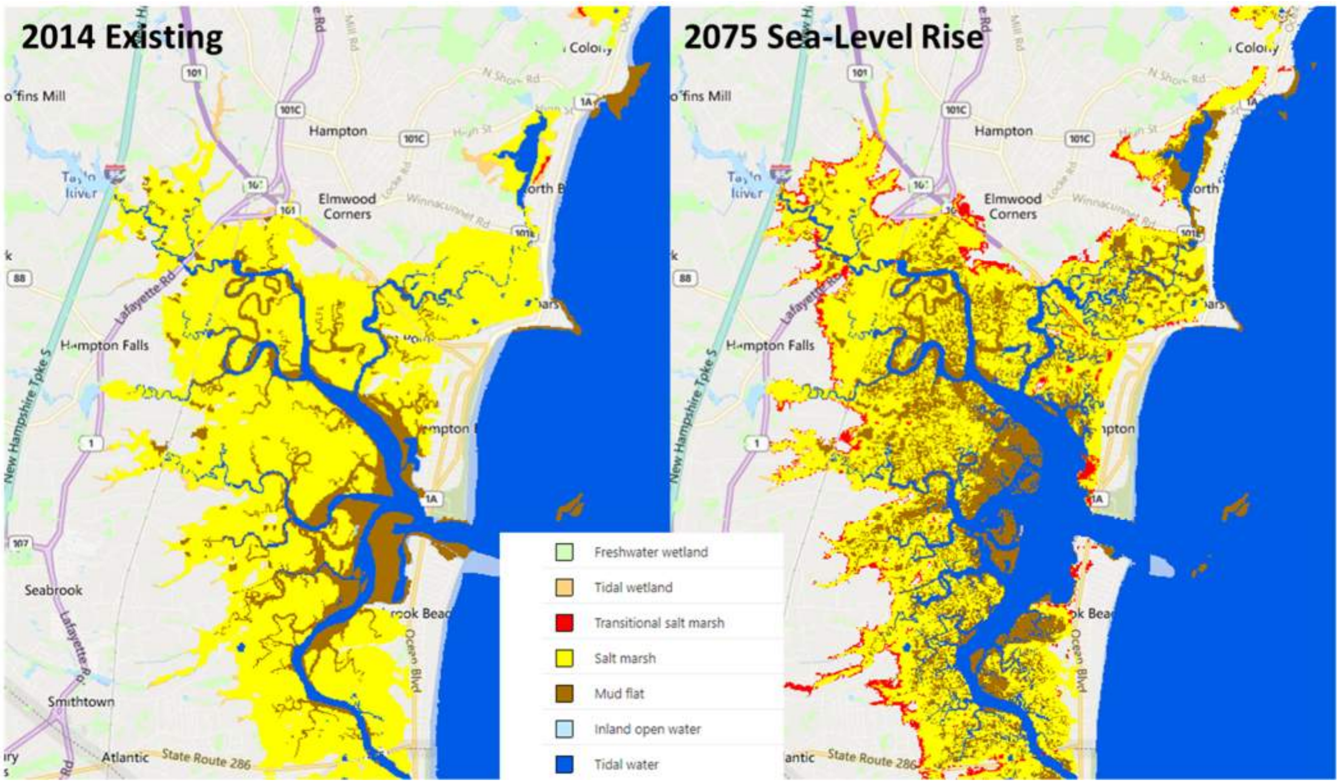


Figure 17 Sea Level Affecting Marshes Model

3.8 GROUND WATER RISE (MMI, 2021)

Groundwater is expected to rise in the future with the sea level in Hampton. For example, a predicted increase in groundwater levels of 0.2 to 0.7 feet is predicted around Meadow Pond if the sea level rises 2 feet (Figure 18). Rising groundwater will influence drainage in the project area and would require water-tight drainage pipes.

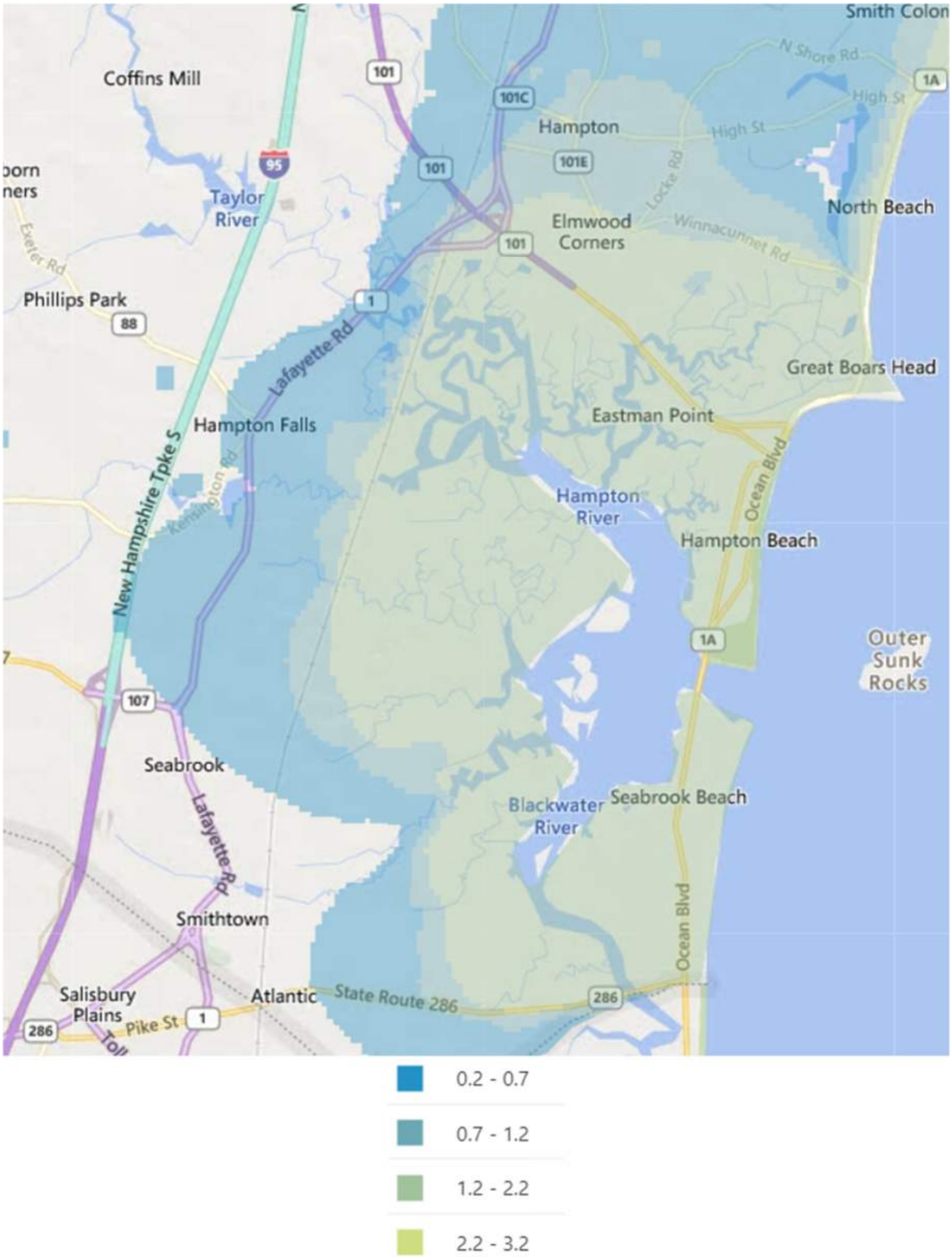


Figure 18 Predicted Groundwater Rise (Feet)

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4. HYDROLOGY

4.0 HYDROLOGY (MMI, 2021)

Hydrology estimates were made for freshwater inflow from upstream watersheds and tidal inflow from the Hampton Harbor.

4.1 FRESHWATER FLOW TO THE HAMPTON SEABROOK ESTUARY (MMI, 2021)

Fifteen (15) watersheds draining to the Hampton-Seabrook estuary were assessed to understand the freshwater inflow to the project area. The inflow channels range in size from the Hampton River (watershed area ~ 16 square miles) to several small unnamed channels (watershed area ~ 0.1 square miles). Flow estimates were initially generated using the USGS StreamStats website and then by scaling from nearby coastal gauges based on guidance from USGS staff.

Peak flow estimates from StreamStats range between 3 and 200 cubic feet per second (cfs) for the predicted 2-year flood and 20 and 800 cfs for the 100-year flood (Flynn, 2003; Olson, 2009). Hydrographs were developed using the NRCS synthetic hydrograph (Dingman, 1994; NRCS, 1997) for the 2-, 10-, and 100-year floods for each of the fifteen freshwater inflows (Figure 19) (Appendix D).

Based on data availability and watershed characteristics, the surrogate New Hampshire Coastal USGS stream gauges used for scaling to in the project area were the Winnicut River at Greenland near Portsmouth (USGS 01073785) (Figure 20) and Berry's Brook at Sagamore Road near Portsmouth, NH (USGS 01073810) (Figure 21).

A flood frequency analysis was performed, and peak flows were scaled from the Winnicut River to each of the fifteen study sites (see Appendix D). Peak flow estimates were variable yet tracked the StreamStats predictions. The results from the flood frequency analysis were ultimately not used given that the 14.1 square mile watershed is much larger than most of the study watersheds other than the Hampton River. Also, the 17-year data record is shorter than recommended for flood frequency analysis.

The 10-year and 100-year flood hydrographs used in the model were thus predicted based on synthetic hydrographs with peaks flows estimated from StreamStats.

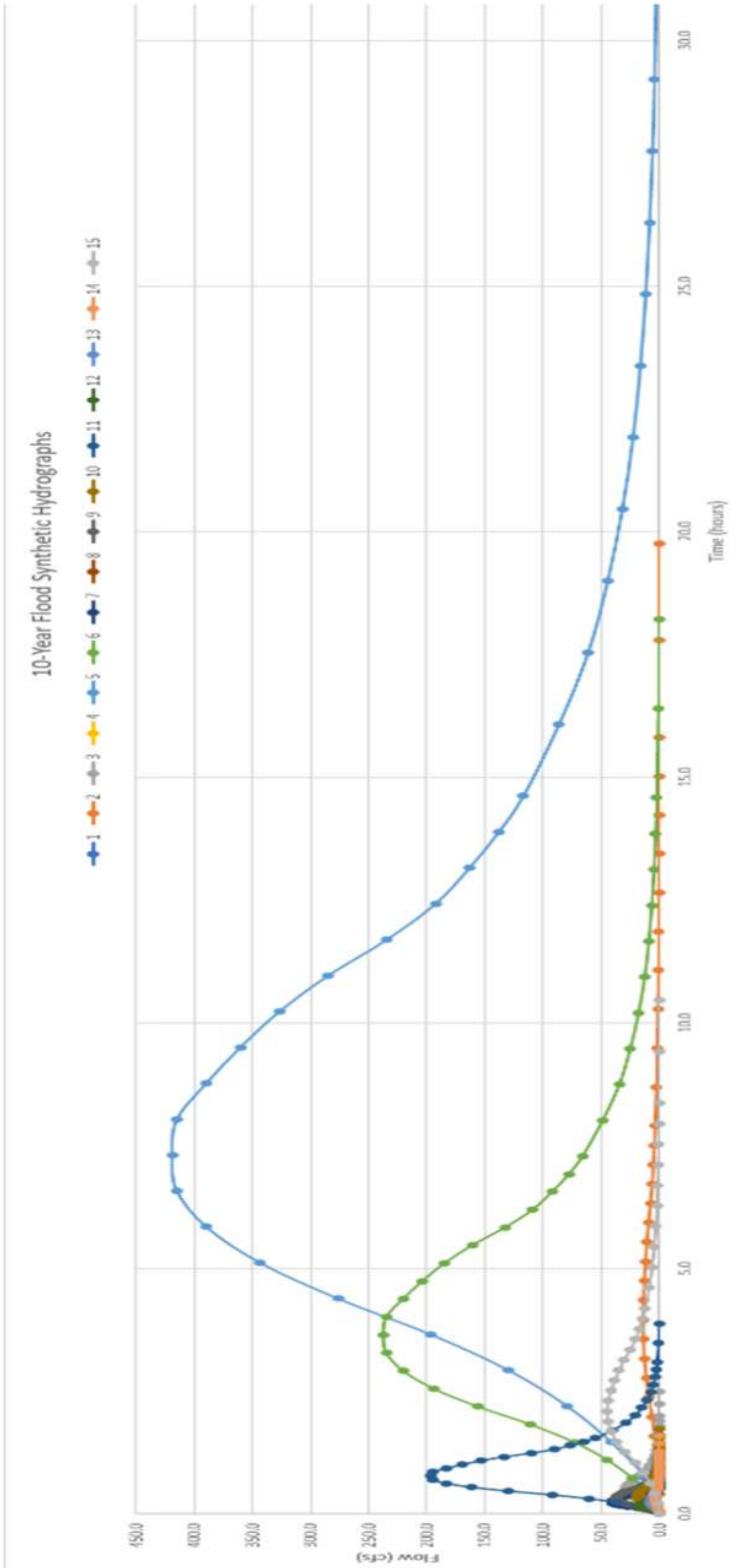

Figure 19 Predicted Hydrographs for the 10-Year Flood

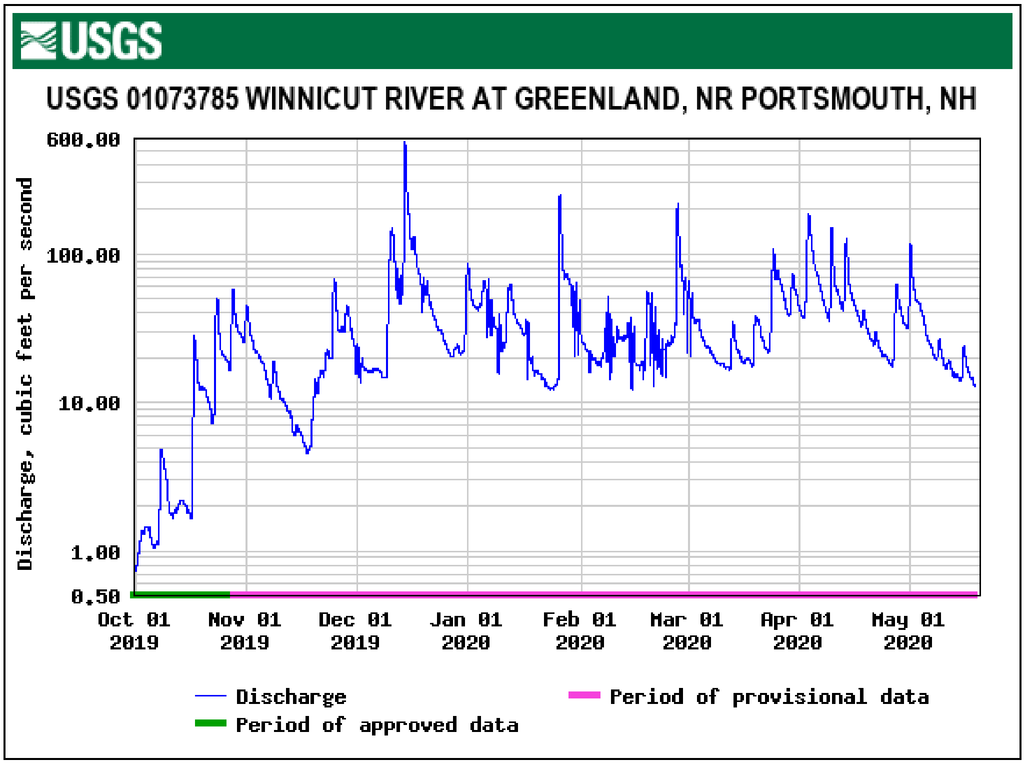

Figure 20 **Sample Surrogate USGS Flow Data from the Winnicut River**

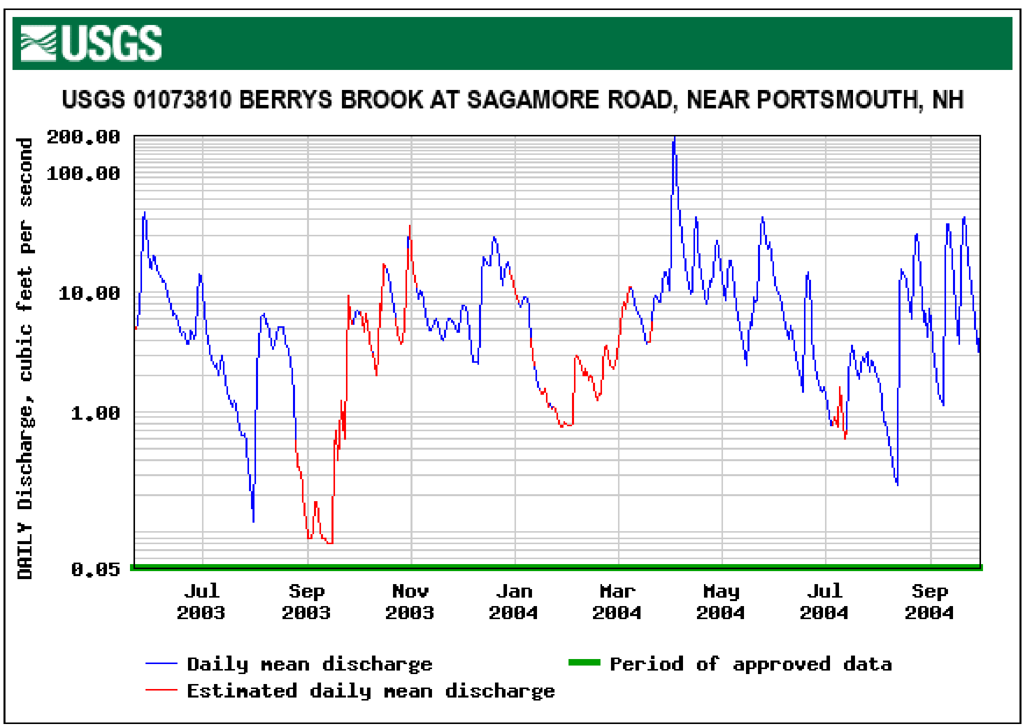

Figure 21 **Sample Surrogate USGS Flow Data from Berry's Brook**

Flow duration analysis was performed for both surrogate gauges (Figure 22 and Figure 23). The analysis confirms a common rule of thumb that the median mean daily flow is approximately 1 cfs per square mile of watershed (1 cfsm). This common relationship was thus used to convert watershed areas to average flow in the project area.

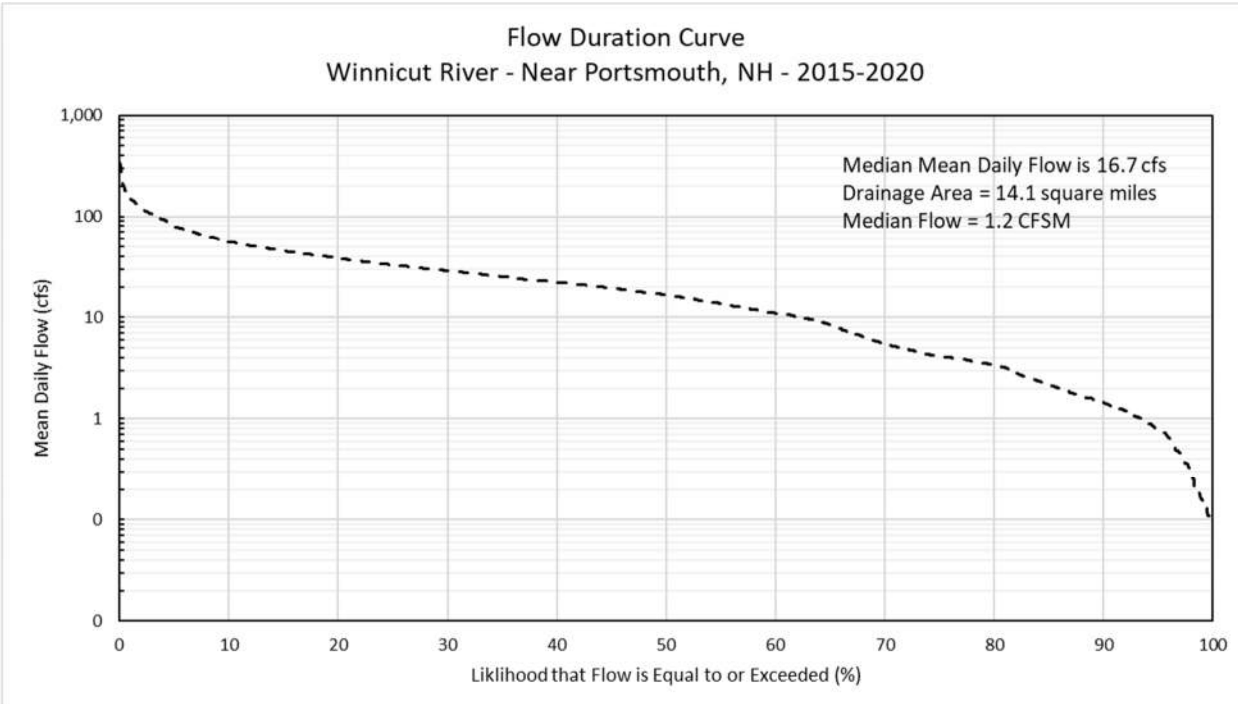

Figure 22 Winnicut River Flow Duration Curve

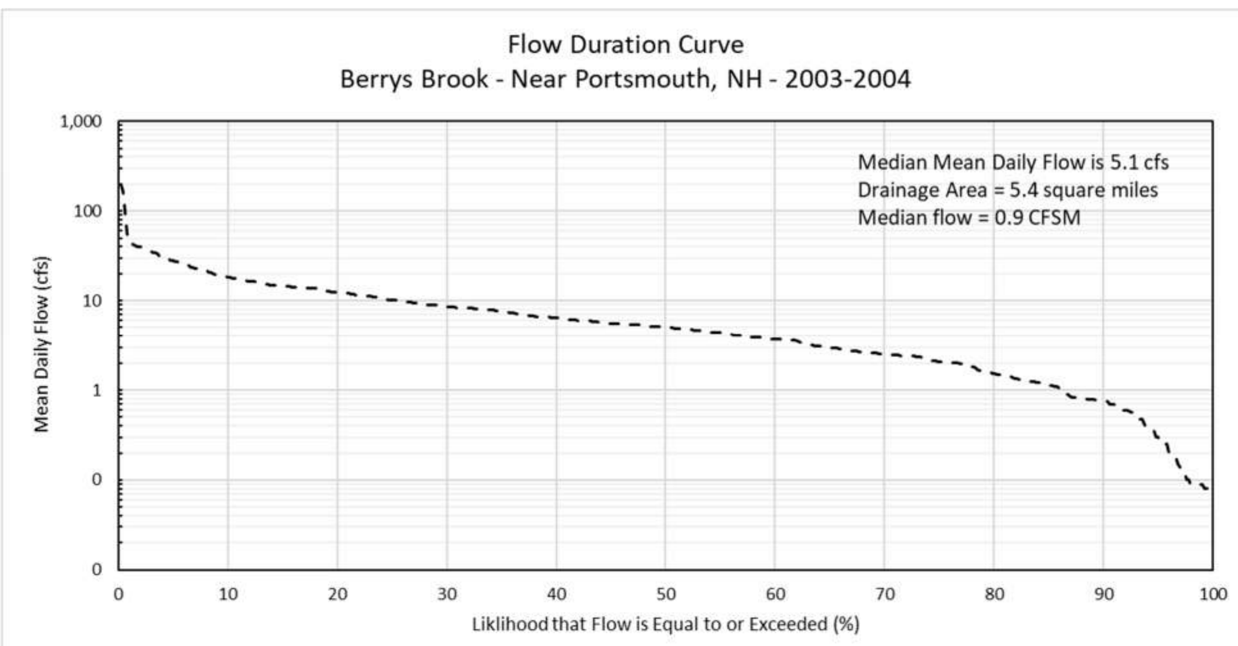

Figure 23 Berry's Brook Flow Duration Curve

During the October 2019 King Tide a small rain event took place along the coast of New Hampshire and the Winnicut River gauge registered a flood of nearly half of the mean annual flood (~ 60 cfs) (Figure 24). Based on the flow duration analysis, this flood had a 13% chance of being equaled or exceeded. This flow normalized by watershed was 3.6 cfsm, and this relationship was used to estimate the freshwater inflow during the King Tide at the project site.

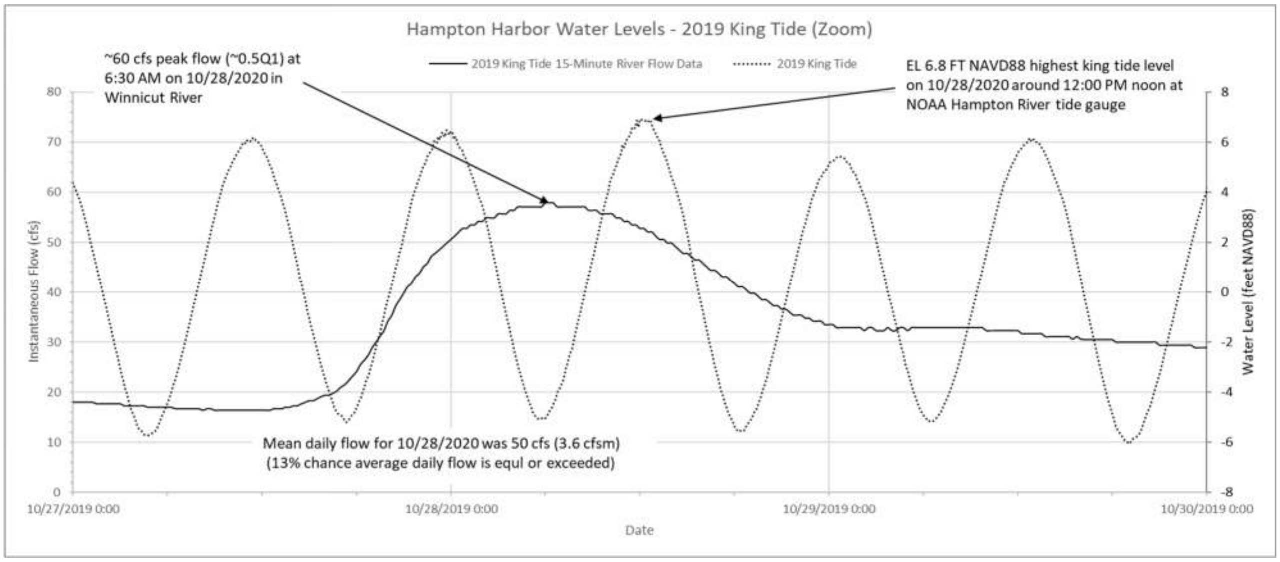

Figure 24 October 2019 King Tide Data and Gauge Data from the Winnicut River

4.2 PREDICTED HAMPTON HARBOR WATER LEVELS (MMI, 2021)

Predicted water levels in Hampton Harbor were obtained from the Woods Hole Group that developed the Massachusetts Coast Flood Risk Model (MC-FRM) that extends into Hampton Harbor (Bosma et al., 2015) (see Appendix D). The water levels include the combined mathematical representations of tides, waves, winds, storm surge, sea-level rise, and wave set-up. Twelve (12) modeled water level scenarios were obtained to understand how water levels in Hampton Harbor may change in the next 50 years. The data reveal increasing tides associated with sea-level rise and changing climate. For example, the 2019 King Tide that caused local flooding in Hampton is like the predicted normal spring tide in 2050 (see Figure 16).

1. Normal spring tide (present day)
2. Normal spring tide (2030)
3. Normal spring tide (2050)
4. Normal spring tide (2070)
5. Astronomical (annual high) tide (present day)
6. Astronomical (annual high) tide (2050)
7. 10-year extra tropical storm (present day)
8. 10-year extra tropical storm (2050)
9. 100-year extra tropical storm (present day)
10. 100-year extra tropical storm (2050)
11. 50-year tropical storm (present day)
12. 50-year tropical storm (2050)

5. HYDRAULIC MODELING

5.0 HYDRAULIC MODELING (MMI, 2021)

A hydraulic model was prepared to recreate existing conditions to understand water depths, velocities, and inundation extents from Hampton Harbor, through Meadow Pond, to High Street. This model was compared to past flooding events and used to evaluate potential mitigation strategies.

5.1 MODEL OVERVIEW (MMI, 2021)

HEC-RAS and RAS Mapper (USACE, 2018) were used to create a two-dimensional, unsteady flow (i.e., flow rate varies with time) hydraulic model of Eel Creek and Eel Ditch (aka Tide Mill Creek) and the salt marsh to investigate flooding. A two-dimensional model was selected for this task given the wide flow area with cross-channel flow and eddying that typically occurs in a marsh setting. The model allows for water to flow in any direction and is therefore able to model tidal flows as they fill and drain from the river system.

The hydraulic model solves equations to determine depth and vertically-averaged velocity across a computational mesh that splits the project area into small regions for calculation purposes. Equations are based on mass and momentum transport equations. This form of modeling allows visualization of flood depths and water surface elevations over the model area. The dynamic model allows for observations of tidal and freshwater movement to investigate constrictions and tidal lag.

The model domain extends south through Hampton Harbor and includes the estuary to the south and west of the Harbor. These areas were included to accurately simulate all inflows and tidal influence that influence the Study area in the northern portion of the model domain.

5.2 MODEL SETUP (MMI, 2021)

A surface of the channel bottom, Meadow Pond, and the marsh were created using the survey data. The surface was imported into RAS Mapper to define the terrain within the immediate project area and channel. LiDAR data were imported into the HEC-RAS model and combined with the survey data to create a digital terrain of the channels and floodplain (Figure 25). The model covers an area of 6,000 acres (9.4 square miles). The model extends 3.5 north of the Hampton Harbor Bridge and includes an additional 2.0 miles to the south of the Hampton Harbor Bridge to include the remainder of the estuary. The model width is 0.6 to 2.3 miles in the east-west direction.

A mesh was created to define the cells where the hydraulic modeling computations are performed (Figure 26). The mesh was created using 30-foot by 30-foot cells. Cell alignment was refined with breaklines to align cell faces to linear features such as road embankments and river channel edges. Crossing structures were modeled using refined geometry in the mesh and the Winnacunnet Road crossing was input as a crossings structure using the 2D connection geometry to represent field conditions.

The hydraulic roughness that is used by the HEC-RAS model to estimate the resistance to flow was developed using GIS land cover data (see Figure 2). A roughness coefficient was assigned to each land cover type (Figure 27).

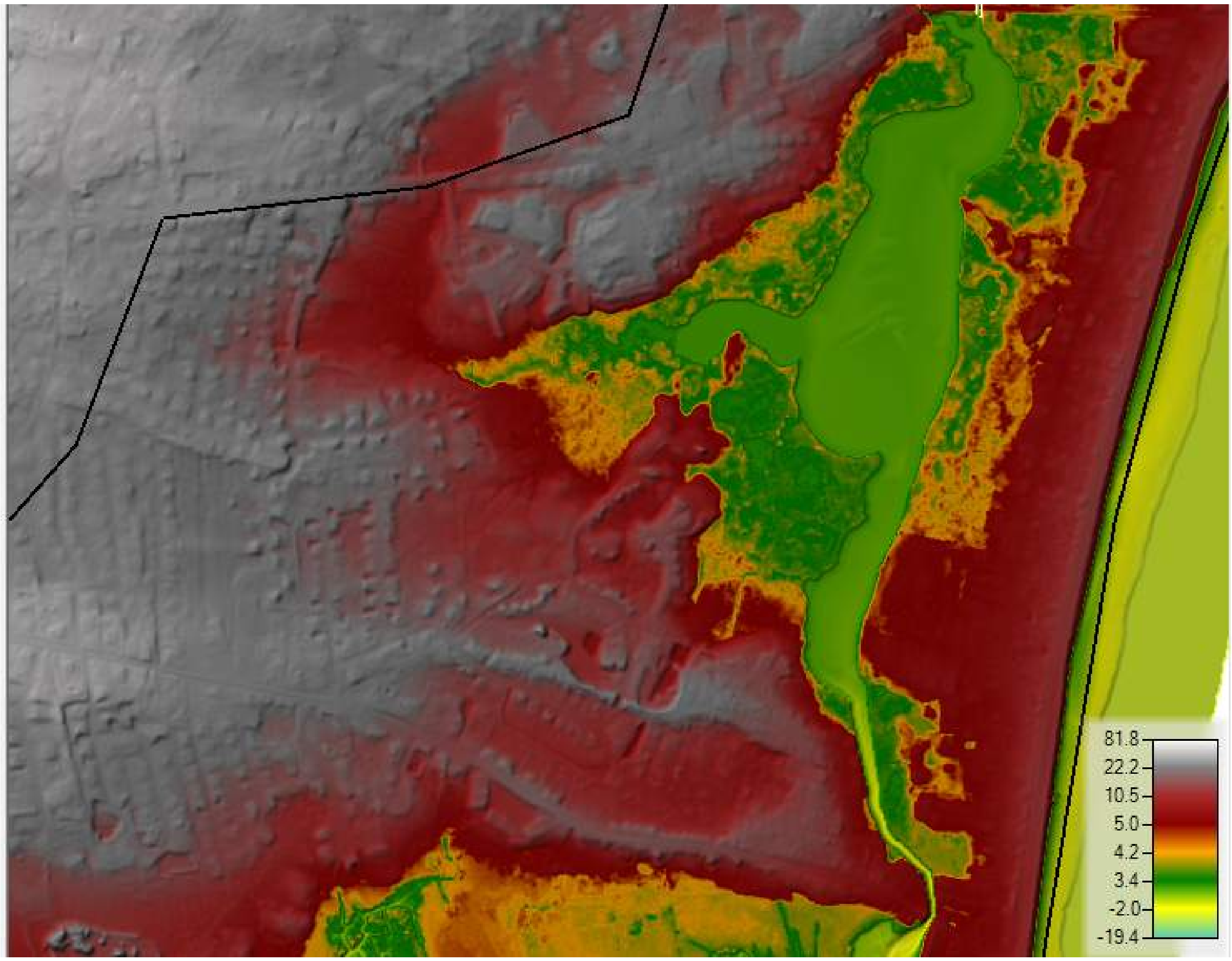

Figure 25 Sample Combined Terrain using Survey and LiDAR Data

Figure 26 HEC-RAS Computational Mesh and Break Lines

The starting water surface elevation for each model simulation was set to 3 feet NAVD 88 based on the typical water surface elevation during low tide from the UNH water level monitoring data.

Data were output from the model every 12 minutes a 36-hour simulation time. The computational time step was set to 3 seconds to stabilize the model.

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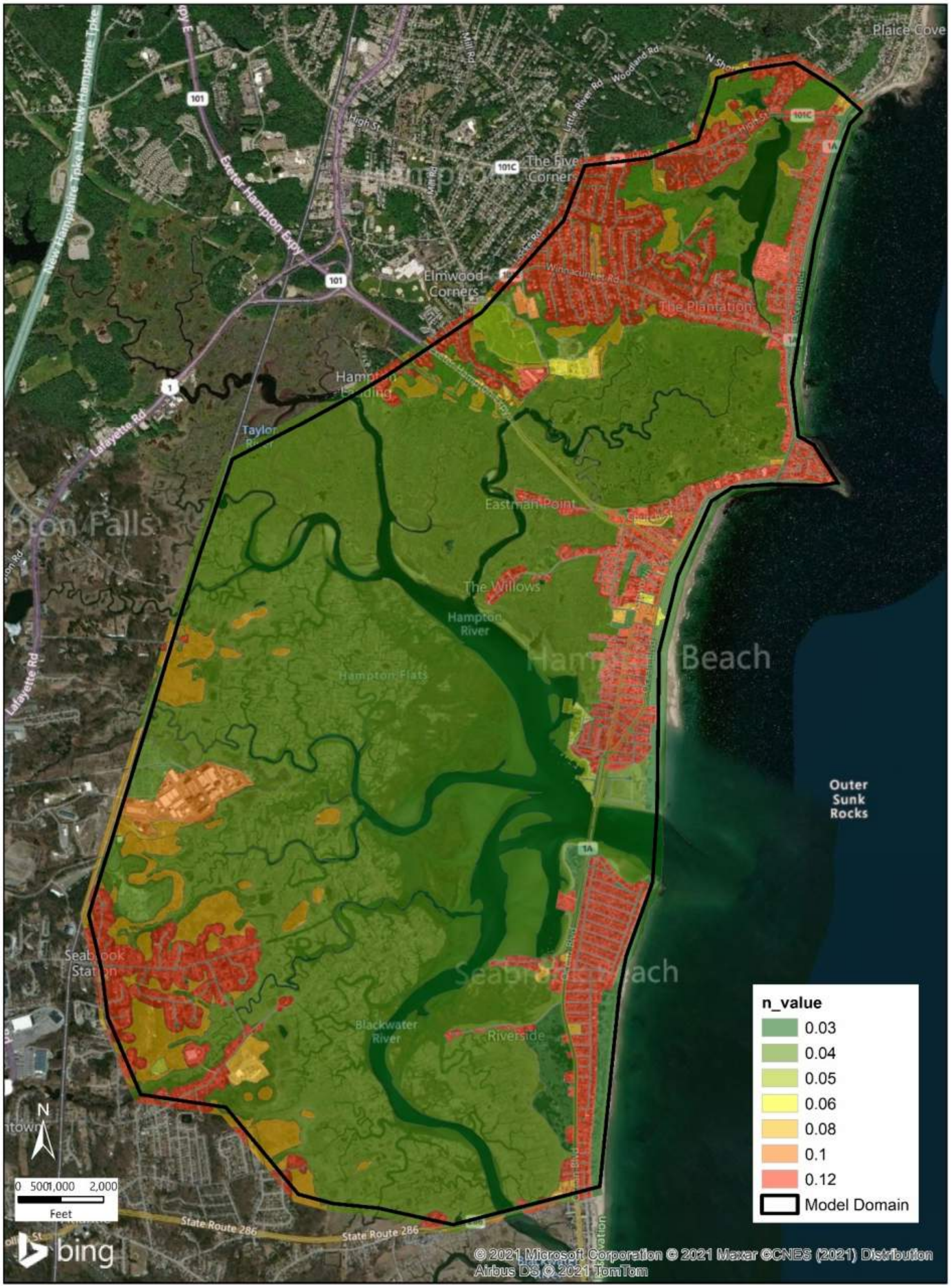

Figure 27 Manning's N Roughness Values Based on Land Cover

5.3 BOUNDARY CONDITIONS (MMI, 2021)

Combinations of upstream inflow and downstream tide were established for the boundary conditions of the hydraulic model (Table 6). The boundary conditions were developed with the project team to represent a range of conditions to explore chronic tidal flooding, extreme tidal flooding, and storm-based flooding. The boundary conditions allowed for prediction of how hydraulic condition is likely to change in the future.

Table 6
Summary of Hydraulic Model Flow-Tide Scenarios

Model ID	Upstream Inflow	Downstream Tide
BC4	100-year flood	Modeled spring tide (2008)
BC6	10/28/2019 estimated small flood	10/28/2019 NOAA king tide data
BC7	Estimated median daily flow	Modeled spring tide (2008)
BC8	Estimated median daily flow	Modeled spring tide (2050)
BC9	Estimated median daily flow	Modeled spring tide (2070)
BC10	100-year flood	100-year extra tropical storm (2050)
BC11	100-year flood	50-year tropical storm (2008)

Three steps were performed prior to entering the boundary conditions into the hydraulic model. (1) Flow hydrographs were interpolated in a spreadsheet to align the timestep with that of the tide. (2) A warmup period was added to the hydrographs to stabilize the model. (3) The timing of the peaks of the freshwater flood and tide were then adjusted to align the peaks for conservative modeling.

5.4 MODEL VALIDATION AND CALIBRATION (MMI, 2021)

The hydraulic model was calibrated against collected water level data and feedback from the Town on typical flood patterns and those observed during the October 2019 King Tide. The initial model results largely reflected flood patterns in the system, yet several locations were identified by the Town where the predicted flood water surface elevations were too high. The model was calibrated by increasing the hydraulic roughness to the higher end typically used in salt marshes ($n = 0.06$) and then the model results generally tracked the expected flood patterns (Figure 28, Appendix E). The model accuracy is estimated to be 1.0 to 1.5 feet.

Inflow from drainage areas 4, 8, and 10 were removed to improve model stability [see Appendix D]. These flow inputs originate from overland flow in developed areas in small drainages with area less than 0.2 square miles.

Figure 28 Maximum Depth Hydraulic Model Results for the 2019 King Tide

5.5 MODEL RESULTS FOR HARBOR AREA

From discussions with MMI and their review of the existing conditions for the Meadow Pond area, it was determined that the freshwater flooding had minimal effects on the water surface elevation in comparison to the effects of the tide, especially in the Harbor area south of the NH Route 101 bridge. Additional information about the existing conditions results is included in MMI's Hampton Flood Mitigation Analysis Engineering Report (2021). Therefore, the existing conditions model results for the Harbor area was reviewed and analyzed for the 2008 Spring Tide (BC7), the 2019 King Tide (BC6), the 2050 Spring Tide (BC8), the 2050 Extratropical Storm (BC10), and the 2070 Spring Tide (BC9).

"It is important to note that this modeling does not account for shallow flooding due to poor local drainage" (MMI, 2021). It also does not account for any splashover that might come from the east side along the beach.

The results generally reflect the existing flooding experienced by residents for current conditions, and the future year results are similar to expectations based on the anticipated sea level rise. Table 7 lists the maximum water surface elevations recorded at the Harbor tide gauge since 2013, as well as the water surface elevations determined through hydraulic modeling at the gauge location. The water surface elevations are typically higher at the mouth of the Harbor and decrease gradually inside the Harbor moving away from the mouth (see Figure 29 and Table 8).

Table 7
Water Surface Elevation (NGVD 88*, feet)

Event	Max
2008 Spring Tide Model	4.80
2013 Max High Tide	6.58
2014 Max High Tide	7.14
2015 Max High Tide	7.23
2016 Max High Tide	6.50
2017 Max High Tide	6.95
2018 Max High Tide	8.04
2019 Max High Tide	6.80
2019 King Tide Model	6.48
2020 Max High Tide	6.60
2050 Spring Tide Model	7.20
2050 Extratropical Model	10.90
2070 Spring Tide Model	9.15
100-Year Flood Stillwater Elevation (FEMA, 2021)	8.36
Typ. Base Flood Elevation of Study Area (FEMA-NFIP, 2021 Jan.)	9.00

*Add 5.2' to convert to MLLW

Figure 29 2070 Spring Tide Projected Existing Conditions

Table 8
Model Water Surface Elevations (NGVD 88*, feet)

Event	Harbor Entrance	Police Station/ Brown Ave	Between Island Path & Glade Path	North of 101 at Water Tower
2008 Spring Tide	4.80	4.77	4.50	4.31
2019 King Tide	6.48	6.51	6.51	5.70
2050 Spring Tide	7.20	7.19	7.07	6.27
2050 Extratropical Storm	10.90	10.94	10.94	10.88
2070 Spring Tide	9.15	9.15	9.09	8.39
2070 Spring Tide with Elevated Roads	9.17	9.19	9.05	8.33
2070 Spring Tide with Elevated Roads & Walls	9.20	9.24	9.16	8.24
FEMA Base Flood Elevation (FEMA-NFIP, 2021 Jan.)	9.00	9.00	9.00	9.00

*Add 5.2' to convert to MLLW

Water depths for the five existing conditions scenarios were reviewed along Glade Path, Island Path, and Ashworth Avenue. The depth of flooding is variable along the roads (due to the variable elevations of the roadway) and dependent upon the flood event. The maximum depths of flooding for each road, obtained for each of the five existing condition models, are summarized in Table 9.

Table 9
Max Water Depth on Roadway (feet)

Event	Glade Path	Island Path	Ashworth Ave
2008 Spring Tide	0.0	0.0	0.0
2019 King Tide	1.4	0.8	0.7
2050 Spring Tide	2.0	1.4	1.4
2050 Extratropical Storm	5.8	5.2	5.2
2070 Spring Tide	4.0	3.4	3.4

After reviewing the existing condition results in conjunction with the tide gauge data, it was determined that the proposed alternatives hydraulic model should be based on the 2070 Spring Tide (Figure 29). The Harbor has previously experienced an extreme high tide event that produced a water surface elevation of 8.05’ NAVD 88 in 2018, which is greater than the expected 2050 Spring Tide water surface elevation of 7.20’ NAVD 88 (Table 7). The 2050 Extratropical Storm produces water surface elevations at least 1.75’ higher than the other events (experienced and predicted) and shows extreme water depths over the roads (Table 9); it was decided that it would be impracticable to plan for the 2050 Extratropical Storm. Therefore, the 2070 Spring Tide was used for the modeling the alternatives. Additionally, the 2070 Spring Tide elevations are about 0.15’ higher than the current BFE in the Study area and the proposed modeled alternatives could be considered adequate for flood mitigation against the 100-year flood provided in the 2021 FIS and FIRMs.

The potential effective alternatives are limited for the Harbor area because the water surface elevations are driven almost entirely by tidal effects due to the proximity of the area to the mouth of the Harbor. There are flood mitigation alternatives that may be effective for the areas north of the NH Route 101 bridge that are not effective for the Study area because this constriction creates tidal lag upstream that is not experienced in the Harbor area. The effects and magnitude of the tidal lag are detailed in MMI's Report (2021). The tidal lag for the 2019 King Tide causes a decrease in water surface elevation up to 2.0' between the area south of the NH Route 101 bridge and the Meadow Pond area, as shown in Figure 30. In addition to the NH Route 101 bridge, the water must also pass through the crossing at Winnacunnet Road (location "D" in Figure 30) to reach Meadow Pond. The 2070 Spring Tide tidal lag can be seen in Figure 29 for the Study area; the water surface elevation decreases by only about 0.5' from the mouth of the harbor to the area between Glade Path and Island Path, but the water surface elevation decreases by about 1.2' at the northern side of the NH Route 101 bridge.

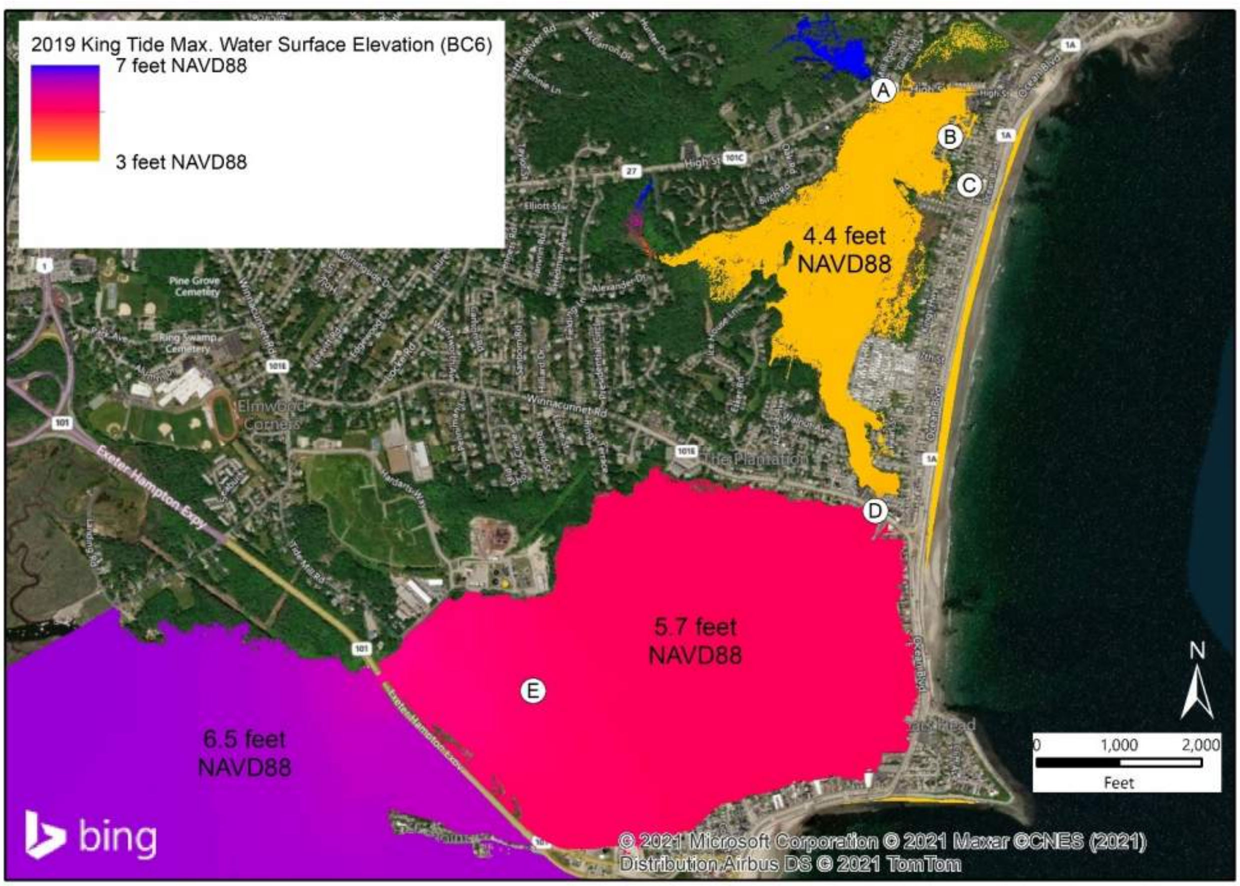

Figure 30 Maximum Water Surface Elevation Hydraulic Model Results for the 2019 King Tide (MMI, 2021)

The most effective flood mitigation solution for the Harbor area is to keep the water out. Therefore, only two mitigation alternatives were modeled in the Study area: elevation of flood-prone roads and installation of permanent barriers. Glade Path, Island Path, Ashworth Avenue, and Mooring Drive were elevated above the 2070 Spring Tide elevation to 10' NAVD 88. Although Ashworth Avenue is not directly adjacent to the marsh area like Glade Path and Island Path, it is the main north-south thoroughfare through the downtown area. Elevating this collector road would improve access and egress from this area during flooding events and therefore is considered in the modeling alternatives. Mooring Drive, an east-west side street between the Harbor marsh and Ashworth Avenue, was modeled to demonstrate the concept of elevating a side road above the design flood event, and to evaluate the potential change in the surrounding water surface elevations. However, due to the density of the buildings on the side streets along the Ashworth Avenue corridor, and the magnitude of elevation change necessary, it would be infeasible to elevate these roadways and the elevated Mooring Drive model demonstrates 'proof of concept' only (see Figure 31 and Figure 43). It is understood that Mooring Drive is already one of the more elevated side streets and that flooding is not currently an issue, but with the increase in water surface elevations due to sea level rise, flooding is expected to be an issue during the 2070 Spring Tide events.

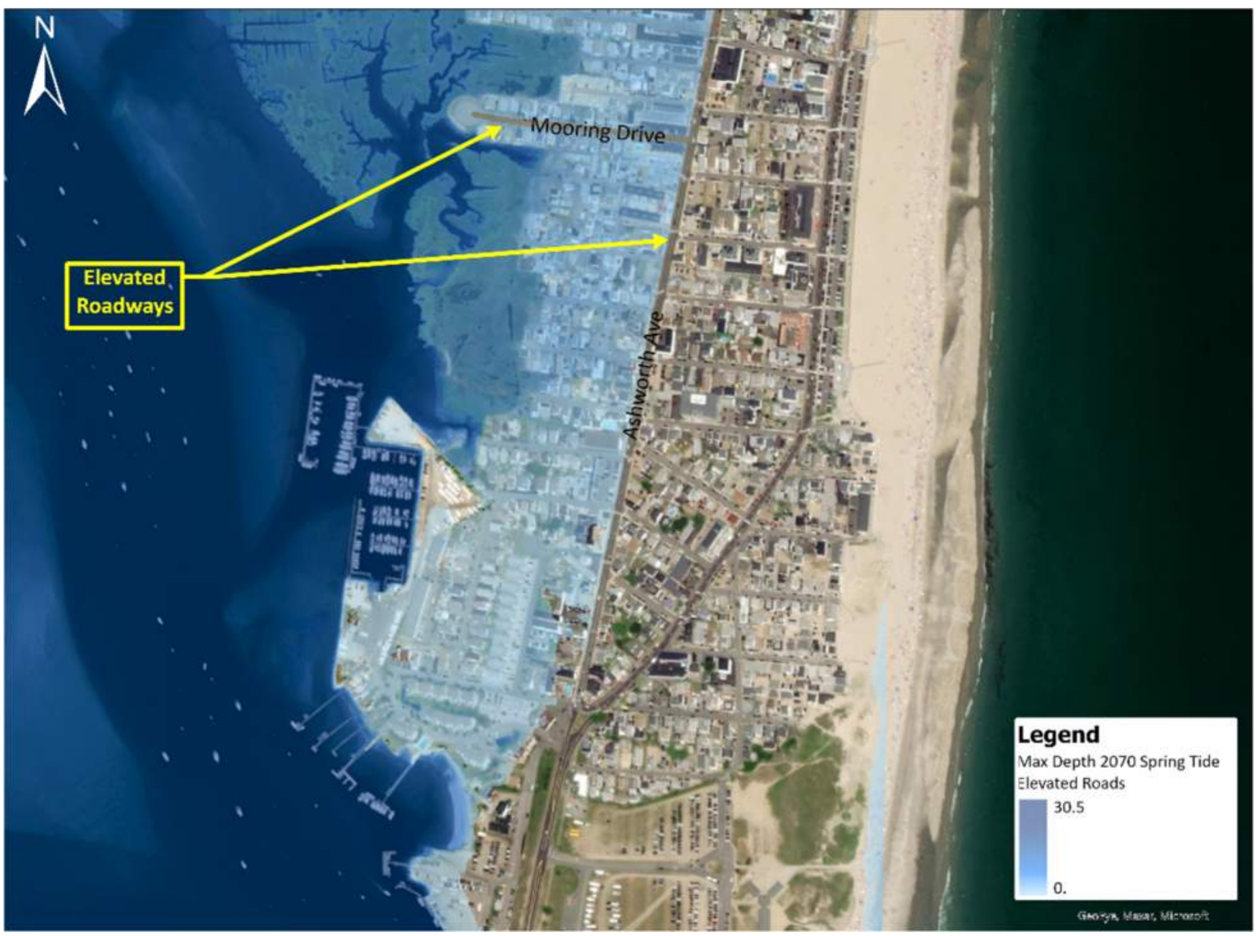

Figure 31 Elevated Roadways Model (South Half) for 2070 Spring Tide

Barriers were added to the elevated roads model to prevent the water from flooding the neighborhoods along Glade Path, Island Path, and Ashworth Avenue (including protecting the side street areas between the Harbor marsh and Ashworth Avenue). It is important to note that portions of the elevated roadways in the model act as barriers and if the permanent barrier alternative was implemented without elevating roads, additional barriers would need to be installed to mitigate the flooding for the entire Harbor area. These walls run along the edge of the marsh to minimize environmental impacts, rather than implementing fewer straight lengths that would shorten the length of the wall, which can be seen in Figure 32 and Figure 39.

Figure 32 Barrier Model (South Half) for 2070 Spring Tide

Both proposed alternative models demonstrate that elevating roads and installing permanent barriers would have minimal effect on water surface elevations; in the immediate vicinity of the infrastructure, water surface elevations increase less than 0.1’ versus the existing condition as summarized in Table 8. The water surface elevation north of NH Route 101 decreases marginally (0.15’ max) due to these changes; this reduction occurs because adding walls and raising roads prevents the flooding across NH Route 101, east of the NH Route 101 bridge, that occurs in the existing condition. Flood water that currently flows from south to north across NH Route 101, near the Brown Avenue / Church Street intersections, is instead channelized through the NH Route 101 bridge to access the salt marsh area to the north.

These results are important because they show that flood mitigation strategies can be used to prevent flooding in the Study area without exacerbating flooding in other areas of the Hampton-Seabrook estuary.

The 2070 Spring Tide has the second highest water surface elevations of the tidal scenarios considered, and results show minimal change in the water surface elevation flood barriers are added and roads are elevated; therefore, it was deemed unnecessary to model additional permutations of the elevated roads and wall alternatives for other storm and tide events.

The model results for the Meadow Pond area are detailed in MMI's Report (2021).

6. FLOOD MITIGATION ALTERNATIVES ANALYSIS

The initial findings of this Study verified findings of our predecessors, that there is no easy solution to eliminate flood risk and damage to existing developed properties adjacent to the Hampton Harbor Area. The findings did uncover a few options that could be implemented to minimize future damage and are summarized below.

6.0 OVERVIEW

Discussion of mitigation strategies focused on “big picture” options and generally fell into three categories: resist the water with infrastructure; accommodate the water with infrastructure; and retreat from the water.

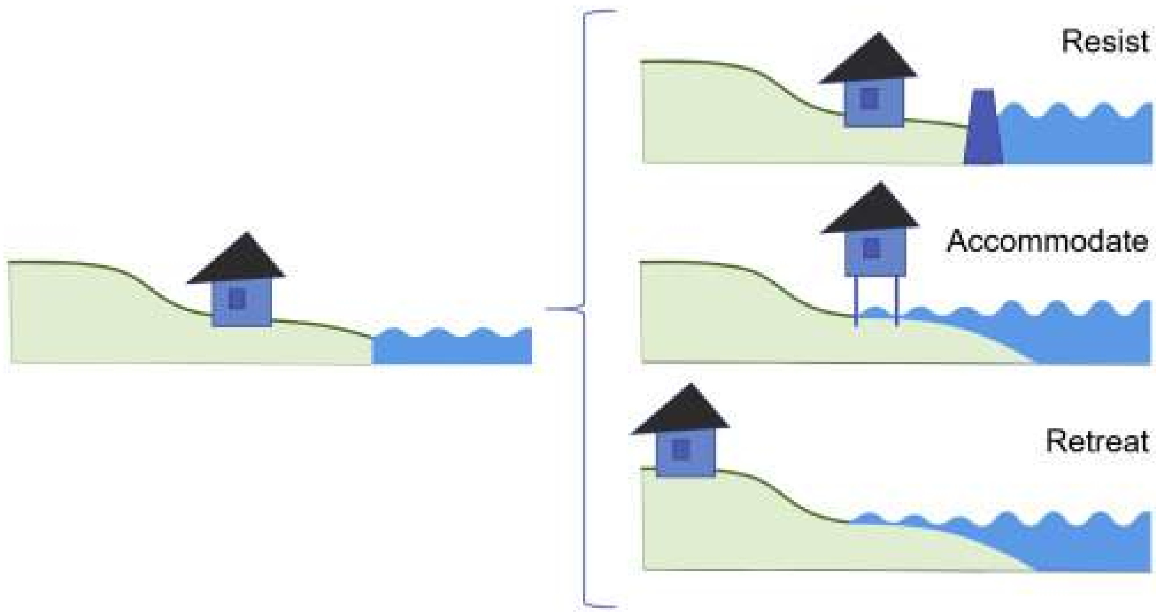

Figure 33 “Adaptation Categories” (Siders, 2019)

Since the Harbor flooding in the areas of interest is caused directly by ocean and tidal impacts, the following strategies that may be effective in areas further away from the mouth of the Harbor (with greater riverine, marshland, and overland buffers), however, were determined to not be viable for the Study area:

- Find preventative flood storage within the watershed adequate enough to hold the volume that would prevent flooding from impacting the area;
- Dredge the marshland, where the area has filled with sediment and phragmites over time, to re-create the prior capacity of the area and prevent water from needing to leave the marshland and flood the neighborhood.

See Section 6.7 for further discussion on why these storage-based strategies are not effective flood mitigation strategies for the Study area.

The following strategies are potentially viable in the areas of interest and, therefore, were considered further in the Study:

- Create a permanent floodwall, levee, or berm (and associated required drainage system improvements) to protect the areas;
- Utilize temporary flood protection barriers to shield the areas;
- Replace/repair backflow devices and tide gates on culverts and other storm drain systems;
- Elevate flood-prone roadways and construct associated required drainage system improvements;
- Revise Zoning Ordinances and/or Site Plan Regulations;
- Elevate flood-prone buildings;
- Relocate the impacted neighborhood areas.

These alternatives are summarized in Table 10 and discussed herein.

Table 10
Harbor Flood Mitigation Alternatives

Category	Subcategory	Description
Resist with Infrastructure	Permanent Barriers	Install walls at low areas of Glade Path
		Install walls at low areas of Island Path
		Install walls at low areas of Ashworth Avenue
		Install walls at low areas of North of NH Route 101
		Vegetated earthen berms
	Temporary Barriers	Walls installed for duration of event and removed
	Elevate Flood-prone Roads	Glade Path
		Island Path
		Ashworth Avenue
Accommodate with Infrastructure	Existing Infrastructure	Elevate flood-prone houses/buildings
		Zoning & site plan regulation changes
		Improve drainage
		Tide gate improvements on stormwater outfalls
Retreat	Property Changes	Voluntary and assisted retreat/relocation

6.1 ALTERNATIVE 1: PERMANENT BARRIERS

6.1.1 DESCRIPTION

The concept of a floodwall, berm, or levee was initially considered for a variety of height, length, and material options. These alternatives were evaluated at a high-level and reduced to consideration of a floodwall consisting of a concrete exterior with a steel sheeting core and foundation, and an earthen vegetated berm with an impervious core. The basic concept for each of these options is that a barrier with top elevation located above the design water surface elevation would be built around portions, or the entire perimeter, of the area to be protected. The wall would be located between the marsh and the neighborhood to prevent water from accessing the neighborhood. During a storm event with rain, water would still collect inside the wall area and potentially flood these areas. Therefore, there would still be a need to replace/repair backflow devices on culverts and other storm drain systems as well as install tide gates on the walls.

It should be noted that there are existing walls that help protect some properties, such as the one shown in Figure 34 that is most likely a segmental concrete block wall. However, these walls are not likely designed and constructed to resist the depth of water considered in this Study, may not be watertight, and do not have a method to prevent water from seeping below the wall and infiltrating the protected area. The existing walls would not serve the same purpose as a permanent barrier installed for flood protection and would likely require retrofit or removal and reconstruction for the barrier strategy to be effective.

Figure 34 Existing Concrete Walls in Hampton

The Study area was subdivided into four areas of consideration for this alternative: Glade Path, Island Path, Ashworth Avenue, and north of NH Route 101. It should be noted that implementation of any mitigation strategy within the NH Route 101 corridor is outside of the scope of this Study, but is necessary to include as the Study area could be flooded from the north if this area is left unprotected (see Figure 35).

Figure 35 Required Barriers Along NH Route 101 Corridor

The permanent barrier alternative was one of the mitigation strategies for which hydraulic modeling was performed for this Study. Modeling of the barrier in all key locations was completed in conjunction with the elevated road alternative, so in addition to providing access through the flooded area, the elevated roads acted as a barrier as well. The 2070 Spring Tide event was used as a basis to determine where barriers are required to prevent flooding in the areas of interest. Other tidal events can be extrapolated using the 2070 Spring Tide proposed analysis and the existing conditions analysis for a given event. Additionally, the existing conditions models can be utilized to determine the additional length of barrier necessary if the alternative were to be implemented without elevating roadways. See Appendix F for the hydraulic model results for this alternative for the 2070 Spring Tide event. Table 11 summarizes the required permanent walls based on the 2070 Spring Tide and assuming approximately a food of freeboard.

Table 11
Permanent Wall Summary for 2070 Spring Tide

Event	Glade Path		Island Path		Ashworth Avenue	North of NH Route 101
	w/o Elev. Roads	w/ Elev. Roads	w/o Elev. Roads	w/ Elev. Roads		
Top of Wall Elevation (ft, NAVD 88*)	10.2	10.2	10.2	10.2	10.2	9.4
Average Wall Height (ft)	3.9	3.3	3.7	4.1	3.5	3.3
Maximum Wall Height (ft)	6.7	6.7	6.4	6.4	6.6	6.6
Total Length (ft)	5100**	4300	7100	4700	8700	2800

*Add 5.2' to convert to MLLW
**~800’ is along NH Route 101

A permanent barrier would require significant initial capital investment, incur long-term maintenance responsibility for the Town, and is likely to face extensive, potentially prohibitive, permitting challenges. The order-of-magnitude cost for constructing a concrete and sheeting wall (Figure 36) is estimated at approximately $20M to $40M based on cost data from recently completed projects, including a similar project in Salisbury, MA that included construction of approximately 3,000 lineal feet of floodwall with a total project cost of $7M. Sections of barriers could be constructed to protect smaller areas if funds are limited, but further modeling would be needed to determine the barrier requirements, and therefore cost, to be successfully implemented along a shorter length.

Factors that influence the potential cost of this alternative include:

- The type of barrier used (i.e. a wall, an earthen berm, or a hybrid thereof);
- Whether barriers are installed as a stand-alone solution, or in conjunction with elevating roadways;
- Freeboard (height above the design water surface elevation). Greater freeboard will prevent or reduce splashover from wave and wind action, but will increase the overall height (and cost) of the wall;
- Extent of storm drainage improvements. Expansion of the existing closed drainage system, installation of stormwater pumping station(s) and tide gates on outfalls, and other drainage improvements necessary to discharge water from within the barrier will be a substantial portion of the cost of this strategy.

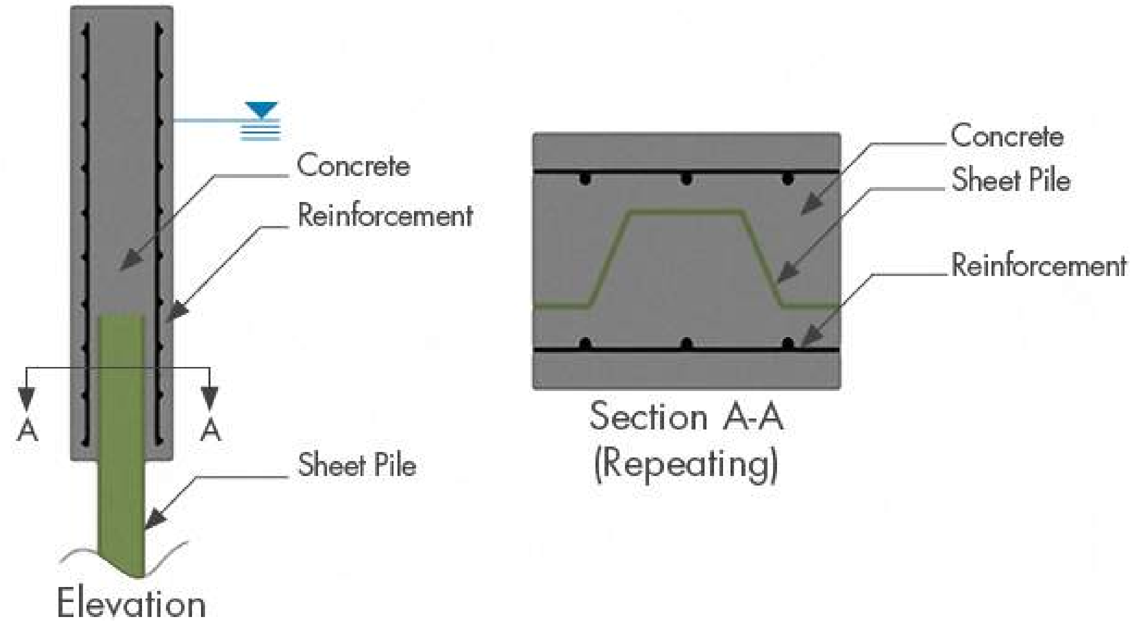

Figure 36 Concrete and Sheeting Floodwall (Kjolsing, 2019)

Other research has estimated that the cost of a concrete floodwall could be as much as 10 times the figures provided above (NACCS, 2015). The initial construction cost for a floodwall per the NACCS Report is about $5,300 per linear foot of barrier, whereas the estimate completed for this Study estimates a cost of $550 per foot. NACCS does state that "a simple steel sheetpile I-wall may be more economical," which is the structure used for this estimate, and that "regional factors, such as materials, labor, and fuel, may affect overall costs" (2015). More refined cost estimating is necessary if this alternative is to be pursued further.

Figure 37 Blackwater River Tidal Estuary Floodwall along 11th Street West, Salisbury, MA (Google, 2021)

It is likely that the cost of an earthen berm would be similar or perhaps greater than a floodwall because of the volume of material required to build the shallow side slopes and the need to install a cut-off wall within the berm to prevent seepage beneath the berm. An earthen and vegetated berm would likely blend with the surrounding marshland better than a concrete and sheeting floodwall, but the wall alternative would have a significantly smaller footprint and therefore less environmental impacts. Both options could obstruct views of the marsh and wildlife. Engineered slope stability solutions could potentially be used to steepen the slideslopes of an earthen berm and decrease environmental impact, but this could increase the cost. This possibility could be further investigated if desired. Most permanent barrier solutions will also require creative design and construction of drainage improvements to reduce the likelihood of seawater breeching the barrier through the existing drainage system.

6.1.2 BENEFITS

There are multiple benefits of a permanent barrier as it greatly reduces flooding. It keeps the water out to protect private property and provides access to the roadways to keep escape routes open and allow for emergency vehicle access during a flood event. This alternative would not be dependent on the residents installing the structures, which would increase the probability of success of the new infrastructure.

6.1.3 CHALLENGES

The primary challenges to any physical barrier strategy are permitting, land availability and required size, viewshed obstruction, and interfacing with existing conditions.

The permitting agencies - NHDES and USACE - have both previously expressed concern that there would be difficulties obtaining permits to move forward with a permanent barrier. Agency coordination and education about model results would need to occur to facilitate obtaining the required permits. Unless it could be demonstrated that there is no need for compensatory storage, it is unlikely that the regulatory officials would issue a federal or state permit for such a structure. Additional documentation and explanation may be needed to show that removing the flood storage of the areas protected by the barrier has minimal effect on the water surface elevation. It can be presumed that the USACE would require an Environmental Impact Statement (EIS) to be developed that would assess the potential for impacts to the quality of the human and natural environments. Development of this type of document could be in the range of several hundred thousand dollars, or higher, and would take several years to complete.

A permanent barrier would be built either on property owned by the Town of Hampton, or on adjacent private property via the purchase of land or the acquisition of permanent easements. It is unknown at this time if the adjacent landowners would be willing to agree to selling land or providing/selling an easement on their land; however, it is anticipated that landowners would be amenable to providing an easement if it was for the purpose of reducing flooding on their property. The barrier along Ashworth Avenue would need to be greater than 1.5 miles long to protect all of the properties in this area. This would be a significant undertaking. Similarly, any berms would require more lateral room to be built due to the shallower side slopes.

Figure 38 Sample Berm/Levee (WorldAtlas, 2021)

There are several docks, piers, and boat ramps into the marsh that would need to be considered with the barrier strategy. The barrier system would need to be designed so that flooding could not short-circuit around the ends and inundate areas intended to be protected. There are flood barriers and flood gates available that could be investigated to help address this issue.

A permanent flood barrier would at least partially obstruct the viewshed for adjacent private properties and public areas and thus may be viewed unfavorably by residents; the magnitude of visual impact would vary depending on the final height of the wall. Further evaluation of a barrier system would have to consider balancing the benefits of increased barrier height (e.g. protection against more severe future flooding events and greater freeboard during current events) versus the greater cost and more significant visual impacts of a taller barrier.

With permanent barriers constructed, additional drainage elements such as backflow valves and stormwater pumps would need to be installed to reduce the likelihood of stormwater using the drainage network to circumvent the barrier, and to remove stormwater that becomes trapped inside the barriers during heavy rain events. Additionally, the existing tide gates would need to be inspected and repaired or replaced if needed, and additional tide gates would need to be installed on any stormwater outfalls of the closed drainage system(s) within the new barrier.

6.1.4 RECOMMENDATIONS

A permanent barrier with associated storm drainage improvements would achieve the project goals of reducing flooding, protecting private property, and increasing public safety, but this strategy presents significant environmental permitting and funding challenges. It is recommended that the Town evaluate whether these challenges make this alternative infeasible, or if it would be worthwhile to further investigate.

Figure 39 Required Barrier Locations for 2070 Spring Tide

6.2 ALTERNATIVE 2: TEMPORARY BARRIERS

6.2.1 DESCRIPTION

Temporary barriers are similar in purpose and function to permanent barriers, but they are installed in preparation for a flood event and removed once the water has receded. There are a variety of options available including NOAQ Boxwall (Figure 40), Inero Flood Panel, water gate flood barriers (Figure 41), and Dam-It Dams, a type of inflatable cofferdam. These come in a variety of heights depending on the owner's needs; for example, the Water-Gate WL height ranges from 6" to 5' tall. Using the estimated depth of water, an appropriately sized barrier could be obtained. Table 9 provides a summary of the maximum water depths on the roadway that could be used to estimate the anticipated depth that could be planned for. These barriers can be installed by property owners (either individually or collectively), the Town, or as a joint effort.

Figure 40 NOAQ Boxwall (Flood Control International, 2021)

Temporary barriers would not prevent all of the flood water from entering into the protected areas, but it would be significantly less water and would slow the flooding down. Additionally, stormwater would still collect inside the protected areas. This alternative would also require replacement/repair of backflow devices on culverts and other storm drain systems to help keep the water on the resident side manageable.

The order-of-magnitude cost for a temporary barrier system ranges from approximately \$60 to \$100 per linear foot of barrier. Assuming protection of an individual single-family residential structure with a footprint of 24' x 36' and a total barrier length of 150' (to provide a minimal buffer around the structure), the initial purchase cost of a temporary barrier system would be about \$10,000 to \$20,000.

6.2.2 BENEFITS

The benefits of a temporary barrier are similar to a permanent barrier, but also include the availability and ease of installation, lower costs, and lack of permitting requirements. The temporary barriers are available in a variety of sizes, so that they can be more manageable and easily available. They are widely available including regional manufacturers.

6.2.3 CHALLENGES

The effectiveness of the temporary barrier strategy is dependent on the timely and proper deployment of the system in advance of the storm or tidal event; depending on the size and complexity of the barrier it could be difficult for some individuals to deploy on their own. If neighbors agree to install a barrier jointly around their group of homes, the installment would be dependent on the reliability of neighbors and assumes they are all in residence at the time of the flood.

Figure 41 Water-Gate WL In-Use (Quick Dams, 2020)

This alternative could be extremely costly if used to protect each individual property (versus a group of properties) and has significant relative cost for a homeowner to buy independently if the temporary barrier is not subsidized. Additionally, the barriers must also be stored when not in use, which could be problematic for some property owners and/or some temporary systems.

As previously mentioned, the protected area would not remain completely dry because temporary barrier systems are installed on the ground surface and are not embedded, allowing some amount of water to migrate beneath the barrier. Depending on the barrier system used and how it is installed, pumps may be needed to maintain a manageable amount of water. Additionally, due to the nature of a temporary barrier, they have a lower factor of safety than a permanent barrier and could be more easily breached than a permanent barrier. It would be much more difficult to keep the roads from flooding with temporary barriers, so emergency responder access would likely remain limited with this strategy.

6.2.4 RECOMMENDATION

Temporary barriers would reduce flooding to help protect private property and increase public safety, while minimizing environmental impacts. This alternative would be simpler to permit and have reduced construction challenges. Although they have a relatively high initial cost, they have a relatively low maintenance cost once purchased. This alternative is viewed as favorable for individual property owners and the Town to consider, especially as a short-term solution. Temporary barriers could be purchased and deployed immediately in areas that a predetermined in order for there to be successful implementation. This could be done alone or in conjunction with other permanent alternatives.

6.3 ALTERNATIVE 3: ELEVATE FLOOD-PRONE ROADS

6.3.1 DESCRIPTION

The elevate flood-prone roads alternative considers increasing the roadway elevation above a chosen water surface elevation. This option would meet one of the goals by maintaining access to residences currently stranded under King Tide and similar high tide storm events. It also has less environmental impact, and less permitting obstacles, than the permanent barrier alternative.

Figure 42 Island Path Aerial During 2019 King Tide (Murray, 2019)

Glade Path and Island Path each experience flooding during most of the scenarios evaluated making the homes to the west temporarily inaccessible until floodwaters recede. Raising Glade Path would provide residents access to NH Route 101 and a means of exiting the area if needed. Raising Island Path would allow residents to leave the immediate vicinity, but without raising additional roadways or implementing other flood control measures, other flooded areas would provide a challenge for complete accessibility.

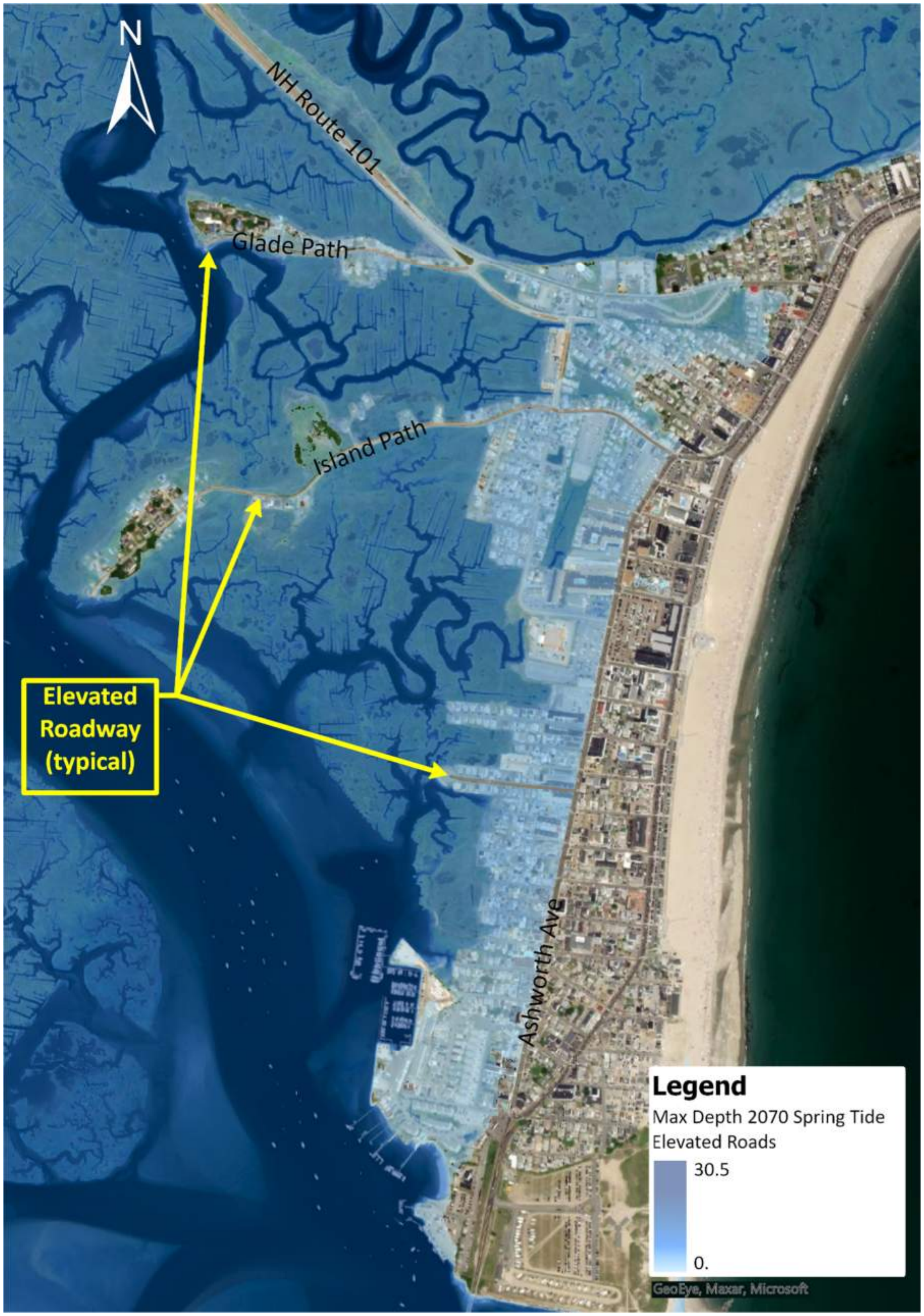

Figure 43 Elevated Roadway Locations for 2070 Spring Tide

Ashworth Avenue extends from the Harbor bridge north approximately 1.1 miles to the intersection with Island Path. The flood scenarios considered for this Study indicate variable depth of flooding on Ashworth Avenue and the intersecting side streets. This area is different from Glade Path and Island Path due to the length of the roadway and the higher density of buildings and intersecting streets on both sides of the road that would be impacted by changing the elevation of the road.

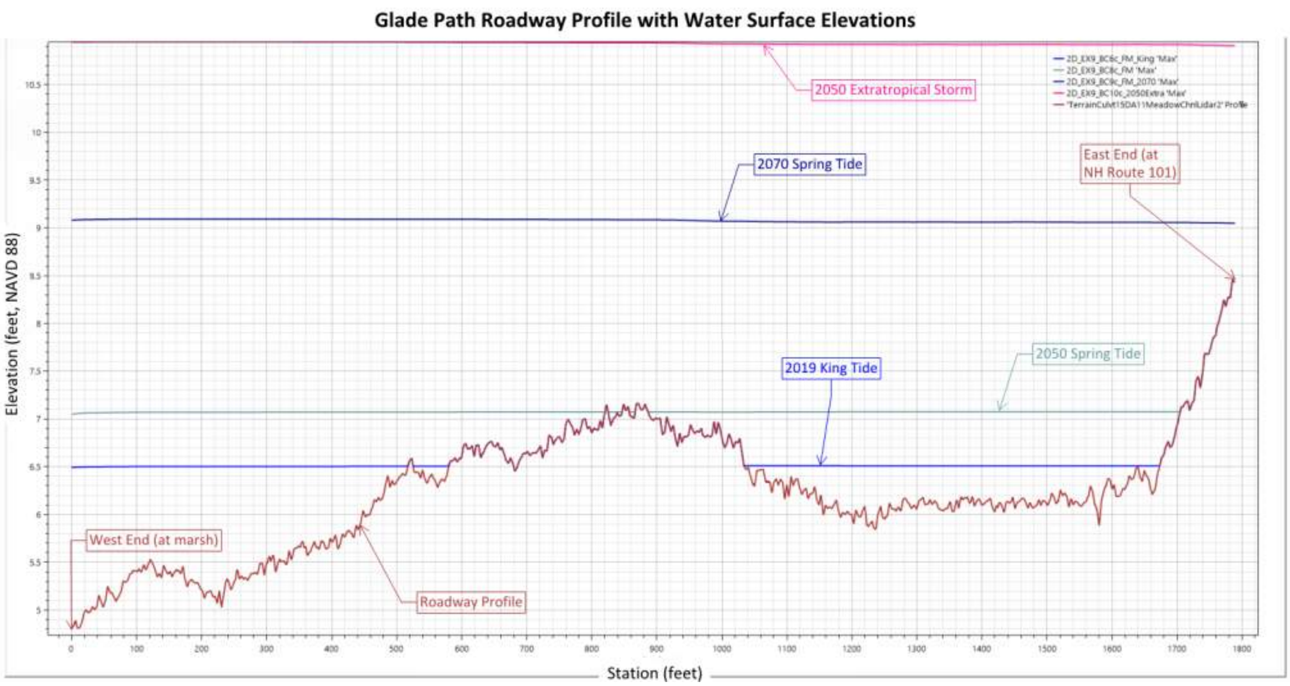

Figure 44 Glade Path Roadway Profile with Water Surface Elevations

Hoyle, Tanner & Associates, Inc.

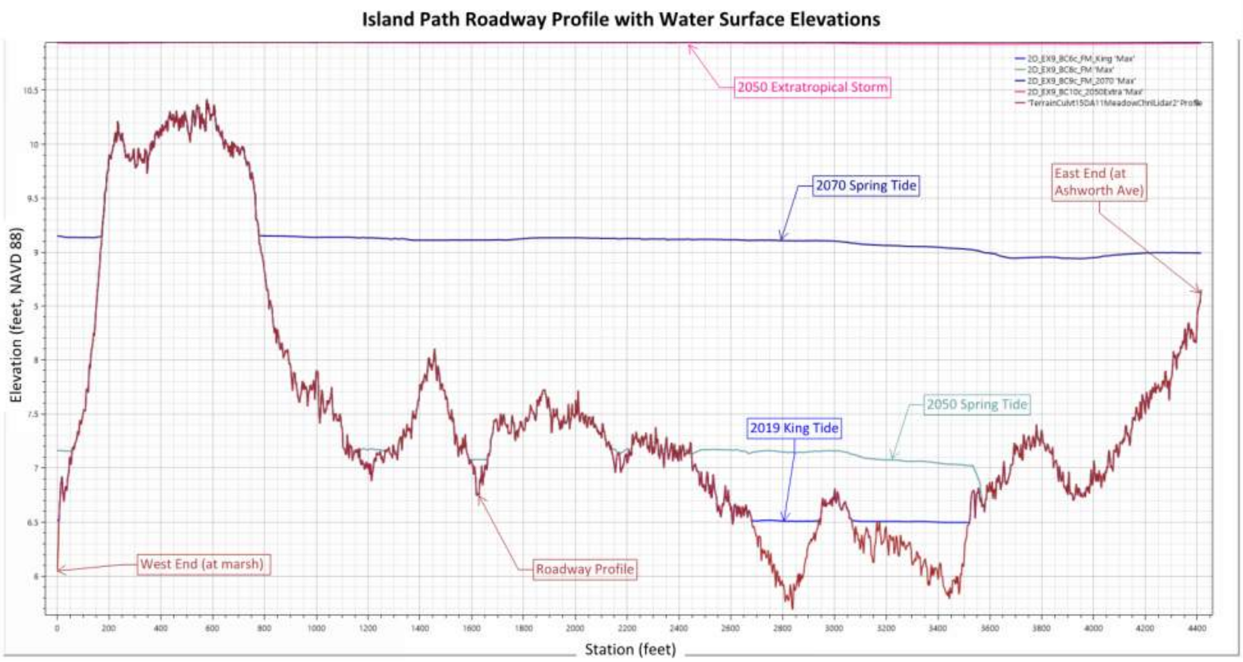

Figure 45 **Island Path Roadway Profile with Water Surface Elevations**

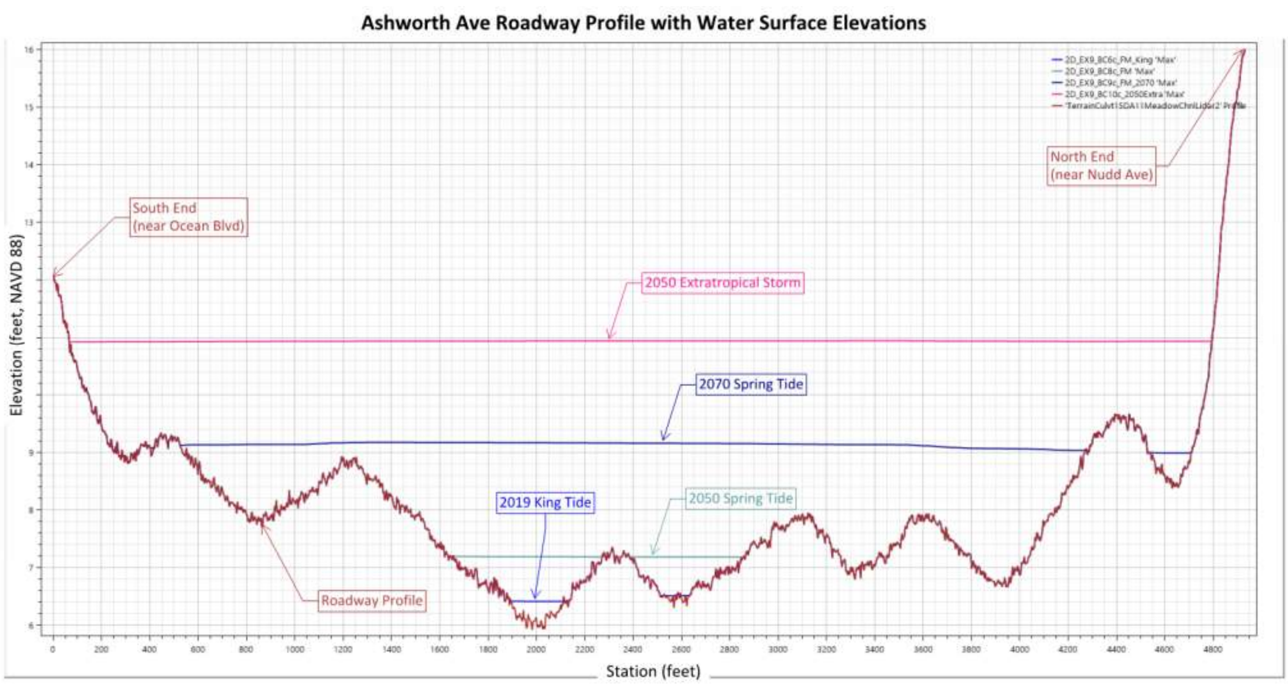

Figure 46 **Ashworth Ave Roadway Profile with Water Surface Elevations**

Roadway delineators marking the edge of the roadway would increase the safety of travelers. This would help drivers stay on the travelway even if the roads have minor flooding, and help prevent them from accidentally driving into the marsh. The delineators should not be taken as authorization to drive through floodwaters that can be dangerous. Signs, such as a permanent yellow warning sign with “When Flooded, Turn Around Don’t Drown®” (Figure 47), reminding travelers of the danger of driving through water should be installed in conjunction with roadway delineators. There are also temporary pink incident signs with “Flooding Ahead Turn Around Don’t Drown®” that could be implemented as well. Even if the roads are not elevated, this is a safety improvement that can be easily implemented at a relatively low cost.

Figure 47 Roadway Warning Signs (Turn Around Don’t Drown®, n.d.)

The cost to elevate inundated roadways ranges from about $0.75M to $1.5M for Glade Path, from about $1.5M to $3M for Island Path, and from about $3M to $6M for Ashworth Avenue. The cost for Ashworth Avenue does not include any costs for elevating any adjacent side roads. The wide range in potential costs for this strategy is due to the unknown nature of the work to tie-in to existing side streets and driveways to accommodate the change in elevations. Some structures have minimal front yard setback from the roadway that would make elevating the roadway difficult without having roadway fill against the structure, so there may be need for property acquisition for this alternative, or the construction of retaining wall systems between the roadway and abutting structure in some areas. Table 12 summarizes the required elevated roads based on the 2070 Spring Tide and assuming approximately 6” of freeboard.

Table 12
Elevated Roads Summary for 2070 Spring Tide

Parameter	Glade Path	Island Path	Ashworth Ave
Approx. Required Top of Road Elevation (ft, NAVD 88*)	9.6	9.6	9.6
Approx. Average Increase (ft)	3.4	2.5	2.1
Maximum Increase (ft)	4.8	3.9	3.7
Total Length (ft)	1800	3800	4100

*Add 5.2' to convert to MLLW

6.3.2 BENEFITS

Elevating the roadways would prevent or limit the depth of flooding on them. This would provide means of egress for residents and maintain access for emergency responders. Elevating roadways could be completed in a phased approach, either by reconstructing individual roadways (or sections thereof) based on priority, or via an incremental approach where the elevation of the same roadway is increased multiple times based on future flooding scenarios. The phased approach also allows additional time for planning, design, funding, and permitting (as may be required) for elevating roadways in challenging areas (such as locations with driveways and/or buildings very close to the existing edge of roadway).

Raising Ashworth Avenue would have minimal environmental impacts since it does not border the marsh and is totally urban/impervious. Raising Glade Path and Island Path would have more environmental impact because the sideslopes from the raised roadways would extend into the marsh. These impacts could be mitigated with retaining walls, but that would substantially increase the cost.

6.3.3 CHALLENGES

One of the most challenging aspects of elevating the roadways is tying them in with existing drives and streets that are not proposed to be elevated to create smooth transitions. Glade Path and Island Path are more constructable due to the reduced number of structures, but both roads have stretches that are bounded by residences that would be challenging to construct. Ashworth Avenue has a few locations that might be feasible to elevate (Figure 48), but overall, it would be the most difficult to construct as there are several side streets and drives connecting to this road (Figure 49). Ashworth Avenue at Figure 48 might be feasible, but the water depth for the 2070 Spring Tide is approximately 1.5’ to 2’ deep at this location, indicating the roadway would need to be raised 2’ to 2.5’ to remain dry. The water depth at the location in Figure 49 is reduced to about 1’, but the bounding buildings and side streets make it very difficult to elevate the road here. It is not feasible to elevate the side streets along Ashworth Avenue because of the denseness of the buildings. It should be noted though, that the modeling shows minimal increase in the flood water surface elevations in the Harbor area if Ashworth Avenue was raised. Each tie-in would need to meet ADA requirements, and ADA accessibility would need to be considered and maintained for all elevation increases. The increased grades of the side streets to connect to the elevated roadways cannot be excessive and site distance would have to be maintained. To successfully tie-in to the adjacent streets and drives, additional engineering would be required as well as cooperation with landowners to facilitate the construction, which would include measures such as elevating their homes.

Figure 48 Ashworth Avenue Street View (Near F Street) Looking North at Feasible Location for Elevating Roadway (Google, 2021)

Figure 49 Ashworth Avenue Street View (Near M Street & Riverview Terrace) Looking North at Challenging Location for Elevating Roadway (Google, 2021)

Another possible challenge would be utilities. If a significant increase in elevation is necessary, it might require that overhead wires be elevated to maintain the required clearance and there are numerous overhead wires crossing the street as can be seen in Figure 48 and Figure 49. Similarly, underground utilities might need to be raised to help facilitate future operation and maintenance on the systems. These could also be affected by sea level rise increasing the elevation and salinity of the groundwater.

6.3.4 RECOMMENDATIONS

Elevating roadways would help reduce flooding and increase public safety to achieve the project goal of maintaining accessibility to the areas, while minimizing environmental impacts. However, it would not protect private property during floods. Elevating portions of Glade Path and Island Path is feasible, and it is recommended to further investigate these options. However, due to extremely challenging constructability issues with the side street and driveway tie-ins, raising Ashworth Avenue is considered not feasible. It would be beneficial to review and coordinate with the Town's Pavement Management Plan for roads that may be elevated, and potentially hold off any improvements until a decision is made regarding elevating these roadways.

6.4 ALTERNATIVE 4: ZONING AND SITE PLAN REGULATION CHANGES

6.4.1 DESCRIPTION

The Town of Hampton participates in the National Flood Insurance Program (NFIP) that provides relief in the form of flood insurance as authorized by the National Flood Insurance Act of 1963. Participants of the NFIP must adopt Zoning Ordinances with standards that meet or exceed the minimum floodplain management requirements (as set for in the Code of Federal Regulations at 44 CFR § 60.3, Flood Plain Management Criteria for Flood-prone Areas). The Town may adopt more stringent regulations as necessary to address local flooding issues.

The Town of Hampton already imposes more stringent construction regulations in the Wetlands Conservation District (WCD). For the Harbor area, the Wetlands Conservation district encompasses the tidal wetlands (see Figure 50) and a 50' buffer beyond the tidal wetland boundary line as explained in the Town of Hampton's 2020 Zoning Ordinance Article II Section 2.3.2. This area lies within FEMA Zone AE, however, the Town requires construction in the WCD to adhere to Zone VE standards. Zoning Ordinance Article II Section 2.3.4.H for Construction Standards for the Tidal Wetland Conservation District, states:

> *New Construction or substantial improvement of any structure including manufactured homes to be placed or substantially improved within the Tidal Wetland Conservation District shall comply with FEMA's Guidelines that the Town has adopted for the VE Special Flood Hazard Area (Section 2.4.11-C Coastal High Hazard Areas (Zone VE) –Construction Standards). The construction work shall have no adverse impacts on adjacent properties. (Adopted March 2019)*

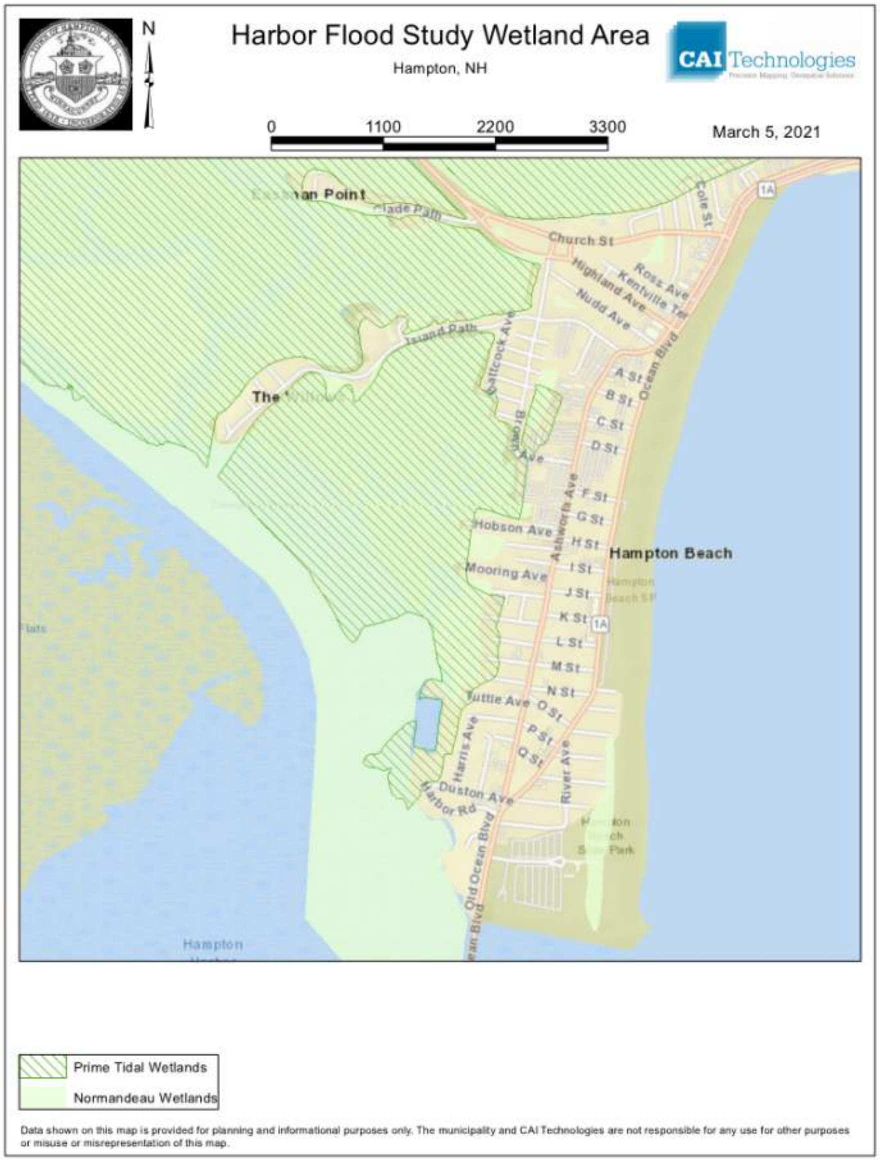

Figure 50 Hampton Harbor Wetland Map

This is a great regulation to adopt, but the WCD does not cover the entire harbor area. It is suggested that the Town adopt these construction regulations for all of Coastal Zone A, as well as some additional regulations as follows.

The Town's Zoning Ordinances include the FEMA requirements for enclosed areas below the lowest floor (Article III Section 2.4.9.D). However, FEMA recommends that buildings in coastal Zone A (including the Study area) meet the Zone V construction requirements, including utilizing open foundations as shown in Figure 51. The Town has already adopted Zone V construction requirements in the WCD, but this area does not include all the structures in in the coastal Zone A. The Town should consider amending the Ordinances to require that all new structures in coastal Zone A, or structures that are elevated to improve flood resistance, be constructed with a pass-through foundation that will allow floodwater to move below the structure during floods. FEMA

Technical Fact Sheet No. 1.2 provides an excellent summary of coastal construction requirements and recommendations (FEMA, 2010) that the Town could utilize in review of their Zoning Ordinances.

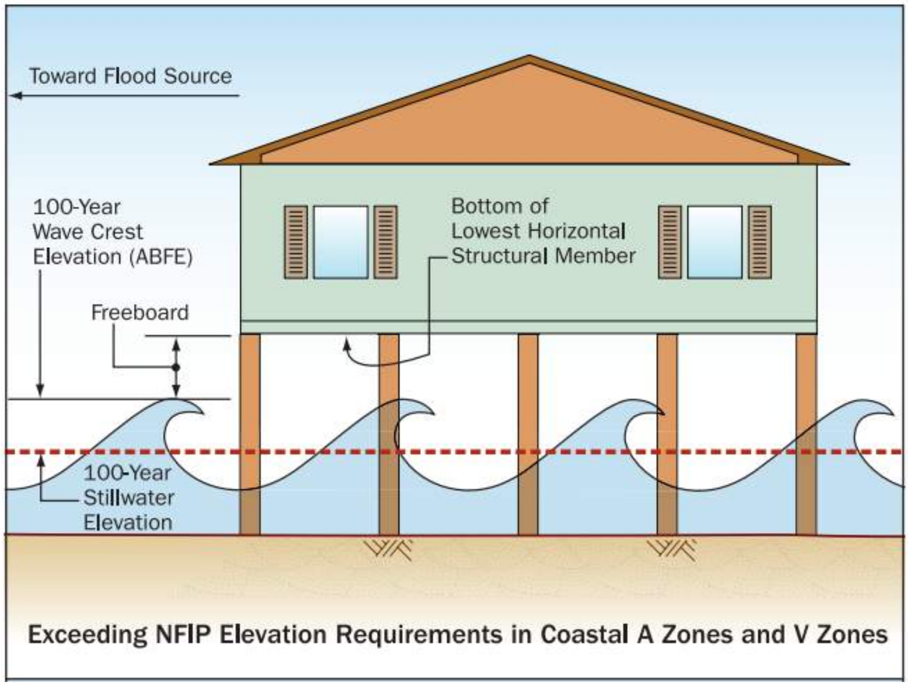

Figure 51 FEMA Recommended Construction in Coastal A Zone and V Zone (FEMA, 2010)

An extreme consideration to prevent flooding of homes and damages would be prohibiting any new construction in the floodplain or limiting improvements to structures that are not raised above the anticipated flood elevations.

One FEMA-mandated regulation is the minimum required elevation for structures built in the floodplain. The Town of Hampton's 2020 Zoning Ordinance Article III Section 2.4.9.A.1, Elevation Requirements states:

> *The lowest floor of a structure, including the basement or crawlspace floor, shall be elevated at least one foot above the base flood elevation (as determined by the Floodplain Administrator in Section 2.4.8). If the elevation of the structure's lowest floor above base flood elevation results in the exceedance of the maximum height requirements (in feet) provided in Article IV, Section 4.4, then the maximum height requirements (in feet) shall be increased by the elevation amount (in feet) that exceeds the maximum height requirement, up to 3 feet. (Amended 2019)*

This requirement exceeds that given in 44 CFR § 60.3 which requires the lowest floor (including basement) be elevated to or above the base flood level (no freeboard required).

Though the Zoning Ordinance requirement includes up to 3' of relief from the Town's maximum building height restriction, owners of existing structures with the lowest floor below the BFE may face challenges in elevating the building to 1’ above the BFE. For residential districts, including the Study area, the maximum height of a building is limited to 35’, or up to 38’ if Section 2.4.9.A.1 applies. It appears that many existing structures could be elevated to 1’ above the BFE without violating the Town’s Zoning Ordinances; however, a detailed review was not performed for this Study. The Town should evaluate if there are existing structures that cannot be elevated to 1’ above the BFE without violating the maximum height restriction, and if revisions to the Zoning Ordinance to provide further relief to those structures may be necessary.

6.4.2 BENEFITS

Revising the Town’s Zoning Ordinances could provide property owners with increased flexibility for elevating their homes out of the flood zone, thereby reducing the risk of flood damage to those structures. This could also streamline the local permitting process by eliminating the need for variances in some situations. Adapting individual existing structures to withstand flooding has less environmental impact than some of the other permanent infrastructure alternatives.

Requiring structures built in Coastal Zone A to meet the criteria of Zone V, including the requirement for pass-through foundations, would increase safety and reduce the potential for flood damage to those structures. These types of foundations protect the structure from dangers associated with scour and erosion of material below the foundation, as well as debris impact.

6.4.3 CHALLENGES

The primary challenge in revising Zoning Ordinances is gaining the consensus required to implement the changes. Property owners near structures that would benefit from relaxed maximum structure height criteria may not support that change because elevating that structure above the base flood elevation could negatively impact their viewshed. Additionally, there is the potential for loss of tax revenue for these viewshed-impacted properties.

Owners of properties within Coastal Zone A are likely to oppose adopting the stricter building requirements of Zone V because it would increase the cost of future modifications to their property. Other residents or Local Officials may also oppose the change because it is only recommended by FEMA (not required) and it could deter property owners from making necessary improvements (i.e. elevating structures above BFE).

6.4.4 RECOMMENDATION

Revising the Zoning Ordinance and Site Plan Regulations could help achieve the project goals of reducing flooding, protecting private property, and increasing public safety, while minimizing environmental impacts. It does not, however, provide a means of access for emergency responders during floods. Whether or not Zoning Ordinances are revised to increase flood protection, it is important to enforce the floodplain management ordinances and reduce the number of variances permitted.

6.5 ALTERNATIVE 5: ELEVATE FLOOD-PRONE HOUSES/BUILDINGS

6.5.1 DESCRIPTION

Buildings in the flood-prone areas can be elevated so that they are above a given elevation for a particular flood event, typically the base flood elevation, which is 9.0' NAVD 88 for the majority of the Study area (see Sections 0 and 6.4 for additional information). This would help protect the building and inside contents including the people. It would not prevent flooding of the land though and other property, such as vehicles, would not be protected.

Historical data for local house construction was reviewed and it was found that the costs for elevating a home spanned from $20K to $200K. There is a wide range due to highly variable nature of the scope of work for each project. It is reasonable to estimate the cost of elevating an individual structure at approximately $150K.

Figure 52 Elevating Hampton House

6.5.2 BENEFITS

Elevating existing structures allows residents to remain in their homes and would reduce property damage. Obtaining permits for this alternative would be easier and the environmental impacts are minimal as compared to permanent infrastructure strategies such as flood barriers and elevated roadways. Although the real estate around some structures is limited due to the proximity of adjacent properties, elevating houses is more constructable and less costly than the other permanent alternatives considered.

6.5.3 CHALLENGES

Although elevating houses would improve safety, it would not provide a means of escape or access for emergency vehicles during high tide flood events. Some areas would be more of a challenge to construct due to the limited space for working, but it would not prevent it from being completed. Another challenge to this alternative is the existing Zoning Ordinances that limit how much a building can be raised while still adhering to FEMA guidelines (see Alternative 4 for more information). This alternative is more of a short-term solution because it does not eliminate nor reduce flooding, and emergency responder access is still restricted.

6.5.4 RECOMMENDATION

Relocating infrastructure from the floodplain is a more feasible option than others that would need to overcome the challenges of obtaining federal or state permitting for construction in the marshland and to identify compensatory flood storage, which is needed for the permanent barrier options. This alternative is a viable solution to help protect properties, but should be considered with revisions to the Zoning Ordinances (see Alternative 4).

6.6 ALTERNATIVE 6: VOLUNTARY AND ASSISTED RETREAT/RELOCATION

6.6.1 DESCRIPTION

Voluntary and assisted retreat and relocation mitigates the risk of flood damage by completely removing buildings and other infrastructure from the floodplain. Flooding experienced during past extreme high-tide and storm events will become more severe in future years; hydraulic modeling predicts that regular spring tide events will inundate most properties on Island Path, Glade Path, Battcock Avenue, and on the side streets to the west of Brown Avenue within the next 50 years (see Figure 53). Though this strategy is likely to face significant opposition from impacted property owners, it should be given serious consideration when evaluating the potential alternatives.

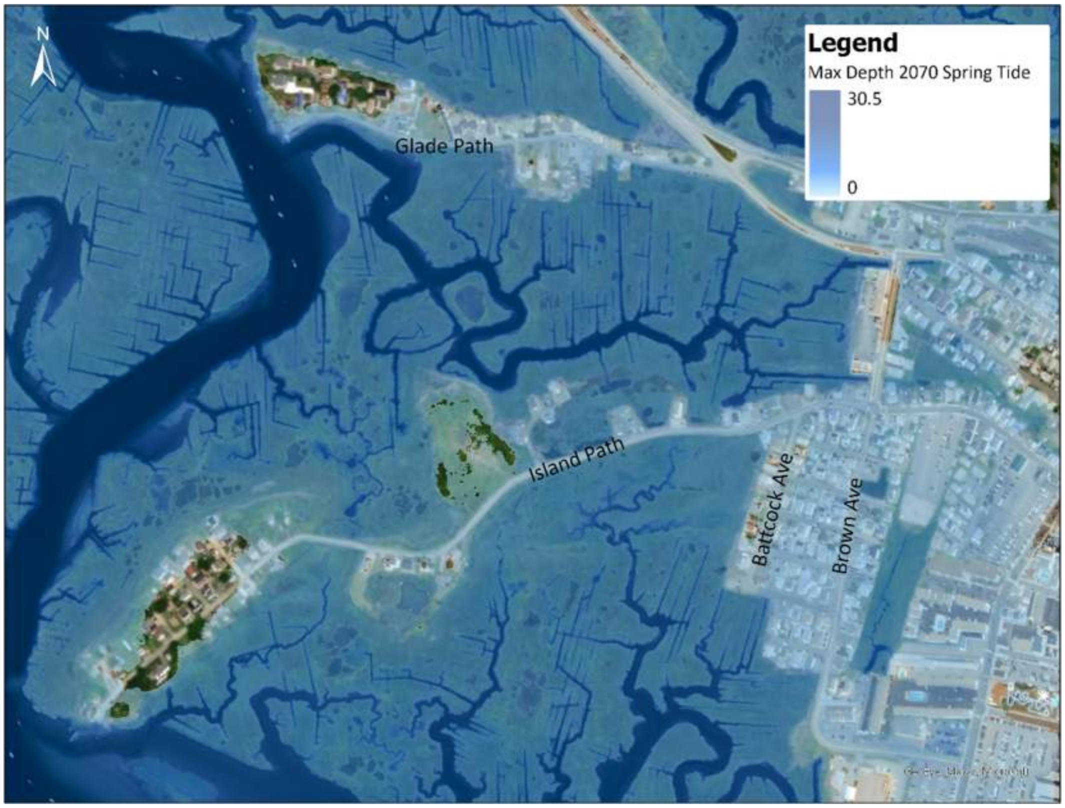

Figure 53 Inundated Areas of Glade Path and Island Path (2070 Spring Tide)

6.6.2 BENEFITS

Relocation out of flood-prone areas is the best way to protect private property because it eliminates the chance of failure. This alternative would increase public safety and may be similar in initial capital investment to some of the other large-scale permanent infrastructure strategies. Relocation has the lowest long-term maintenance cost despite the loss of property tax revenue from the impacted parcels.

This alternative has the least environmental impact of all the strategies, and therefore would be the least likely to encounter opposition from permitting agencies, because it provides an opportunity to repurpose the land and retore some of the marshland. The land could be converted to conservation areas and/or waterfront parks that could be designed to flood and help stabilize the marsh against sea-level rise. Implementation of this strategy could also help position the Town for grant funding that would not be available for the other alternatives.

Removing structures and associated fill and impervious cover from the floodplain provides an opportunity to re-establish some flood storage; however, as previously discussed, given the proximity of the Study area to the mouth of the harbor, the additional flood storage would have minimal impact on water surface elevations.

6.6.3 CHALLENGES

The greatest challenge to this alternative would be convincing the residents to relocate. Most of the residents enjoy their homes on the marsh and most likely would not want to move, even though this would be the safest option for them. This is the psychological challenge that would need to be overcome to achieve this alternative.

The cost of fully implementing this strategy is significant and could exceed $200M based on 2021 dollars and the current real estate market. Assuming an average purchase price of $450,000 per impacted parcel plus $50,000 to restore the land to unimproved status, approximately $20M would be required to relocate all the property owners along Glade Path (~40 properties) and $30M for the owners along Island Path west of Battcock Avenue (~60 properties). The number of impacted parcels along the side streets west of Ashworth Avenue is approximately three times the number along Glade Path and Island Path combined (~300 properties total), resulting in a cost of about $150M to relocate all owners in the flood zone along Ashworth Avenue.

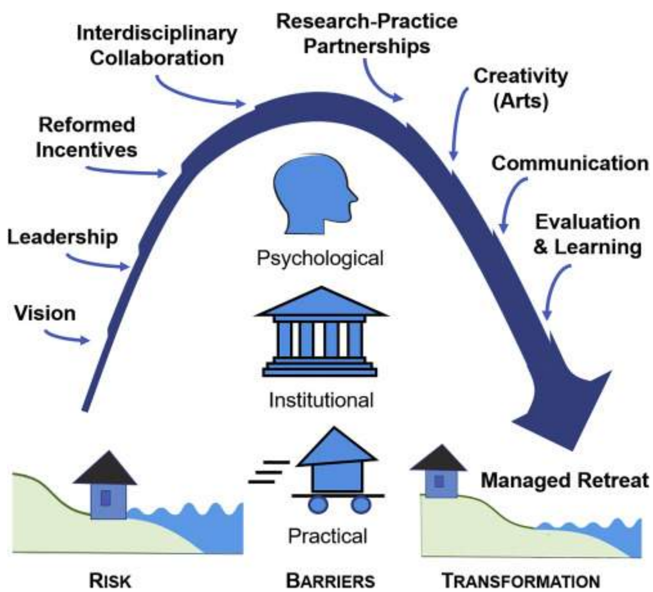

Figure 54 "Overcoming Barriers Requires Diverse Actors Coordinated by Leadership and Vision" (Siders, 2019)

In addition to the initial capital investment to purchase the properties and remove buildings and fill, there are long-term financial impacts from this strategy due to loss of future tax revenue from these parcels.

6.6.4 RECOMMENDATION

Managed retreat including building and fill removal are recommended. Although this alternative is likely to face opposition, particularly from impacted property owners, this is the safest and most reliable strategy. It is also likely one of the more economically viable because there are additional funding sources that could be utilized for this option, including Federal grant programs, that are not available for other solutions due to the nature of the work. It is recommended that managed retreat begin now on a case-by-case basis as properties become available for purchase, landowner willingness increases, and funding sources become available. A formal managed retreat plan should be developed in conjunction with consideration of the other temporary and permanent flood mitigation strategies that the Town will implement.

6.7 OTHER ALTERNATIVES

6.7.1 HYBRID STRATEGIES

The Town is likely to implement a combination of various flood mitigation strategies to balance the benefits and challenges of each alternative and the needs of each area. For example, elevating roadways where feasible and installing temporary or permanent barriers where elevating is not, such as in areas with intersecting roadways and/or high building density. Permanent infrastructure solutions should consider both short- and long-term strategies and be prioritized based on the severity of flooding and the extent of property impacted. For example, it may be prudent to limit the use of permanent infrastructure solutions in locations where managed retreat will be used.

Additional research and refined costs are required though to determine which would be best for each area.

6.7.2 STORM DRAINAGE IMPROVEMENTS

This Study does not recommend pursuing storm drainage improvements as a stand-alone solution to the large-scale tidal-driven flooding within the Harbor Study area. However, the existing storm drainage system may require improvements to address localized flooding issues; therefore, it is recommended that the Town continue to maintain and upgrade their storm drainage infrastructure as necessary to properly handle surface runoff, to prevent localized flooding from tidal inundation of drainage outfalls, and to help mitigate potential effect of sea level rise on groundwater. As previously discussed, tide gates and stormwater pumping stations are recommended in conjunction with other alternatives, such as flood barriers, to discharge water trapped inside the barrier.

6.7.3 CREATION OF FLOOD STORAGE

As previously discussed, the Harbor flooding is due to ocean and tidal impacts. Hydraulic modeling results for the alternative with the greatest reduction in flood storage volume, the permanent barrier, shows minimal impact on water surface elevations throughout the Harbor despite the loss of storage. Addition of flood storage would therefore have a similar result – minimal reduction in water surface elevation and minimal relief from the flooding associated with it. It would be impractical to create the magnitude of storage volume necessary within the Harbor to significantly influence the rising water caused by the tidal boundary condition. Creation of a flood storage area would entail identifying either parcel(s) of land currently outside of the floodplain but easily connected to the area, or parcel(s) that are located in the current floodplain but at an elevation high enough that they are not currently flooded during flood events. In either situation, the storage would have to be located above the normal low tide elevation so it would empty each tide cycle to leave the volume available (empty) for the next tide cycle. The parcels would also need to facilitate a certain depth of dredging above the groundwater table, or the dredging would create a pond. This means the storage area would also have to be shallow and spread over a huge area.

Flood storage volume could be created via dredging areas of the marsh above normal low tide elevation, but the impact would need to be permitted by NHDES and USACE. A dredge effort extensive enough to be even slightly impactful would not be permittable without extensive hydraulic modeling, impact analyses, and likely the development of an EIS. The environmental impacts, including to fish, wildlife, wetlands, and water quality that would result from significant dredging of a very natural and healthy marsh system would be nearly impossible to mitigate. Also of importance to note is that any positive effect a dredge project may have would not be permanent as sediment deposition would be constantly reducing flood storage and future dredge projects may have to be repeated on a periodic basis to maintain effectiveness.

As a result of these considerations, exploring the creation of flood storage volume, particularly via dredging of upper marsh areas, was determined to not be a viable solution and was not investigated further.

6.7.4 HARBOR FLOODGATE

A floodgate installed across the mouth of the Harbor would achieve one of the project goals by regulating tidal flooding within the entire Hampton-Seabrook estuary; however, the cost of this extreme strategy would be tremendous (in the billions rather than millions of dollars) to construct and for that reason alone is considered infeasible for the Town to pursue. Other *nations* have successfully implemented these types of structures, such as the 1.9-mile-long Oosterschelde storm surge barrier in the Netherlands (approximately $7 billion USD in 2020 dollars; Stichting Deltawerken Online, 2004), and the inflatable floodgate system currently under construction in Venice, Italy (about $6.2 billion USD; Wiedeking, 2020); however, the extent and value of the property protected by these elaborate and costly flood control systems justifies their expense. Evaluation of whether a floodgate at the mouth of the harbor could be regionally beneficial, especially with consideration of sea level rise, and financially viable for the entire harbor area is outside the scope of this Study.

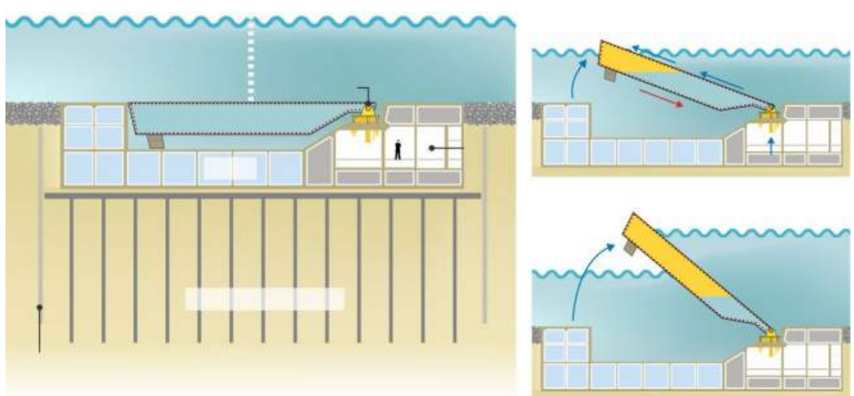

Figure 55 Inflatable Floodgate Concept Utilized in Venice, Italy (Stancati, 2019)

6.8 SUMMARY

The alternatives described above range from easily implementable to infeasible. Combinations of the alternatives may be useful to balance the pros and cons of each one as well as the viability of each one. There are some solutions, such as the temporary barriers and improved drainage, that could be implemented relatively quickly if there is funding. Other alternatives will require additional investigation to determine if they are feasible. It should also be noted that although the NHDES Coastal Program was consulted about the alternatives, their feedback and input are not a final indication of approval or recommendation of any of the options considered.

6.9 RECOMMENDED ALTERNATIVES

No one single alternative is recommended at this time. The most viable short-term solutions are the use of temporary barriers and revisions to the Zoning Ordinances and Site Plan Regulations. The most viable longer-term solutions are elevating key roads to allow for access in and out of the area, and voluntary and assisted retreat. Managed retreat of affected structures out of the flood-prone areas should be seriously considered and prioritized. In order to begin implementation of this option, coordination with FEMA and regular checks need to be made for available and affordable land acquisition options.

It is recommended that further investigation be completed for the permanent barrier and elevated road options to better understand their feasibility, constructability, cost, and potential funding sources. It is also recommended to monitor sea level rise, new technology for flood protection, and other solutions that cities, such as Boston, might implement for flood control. Hampton is not the first or only community to deal with this issue and experiences of other communities might be valuable in deciding which strategies to implement. The following reports are valuable references for previous studies and reports done on in the region:

> *Integrated Analysis of the Value of Wetland Services in Coastal Adaptation; Methodology and Case Study of Hampton-Seabrook Estuary, New Hampshire* by Paul Kirshen dated January 31, 2018;
>
> *Climate Resilient Design Standards & Guidelines for Protection of Public Rights-of-Way* by the Boston Public Works Department dated October 17, 2018;
>
> *North Atlantic Coast Comprehensive Study: Resilient Adaptation to Increasing Risk* by USACE dated January 2015.

The following actions are recommended to implement, or further evaluate, the alternatives found to be potentially viable for mitigating flooding in the Harbor area:

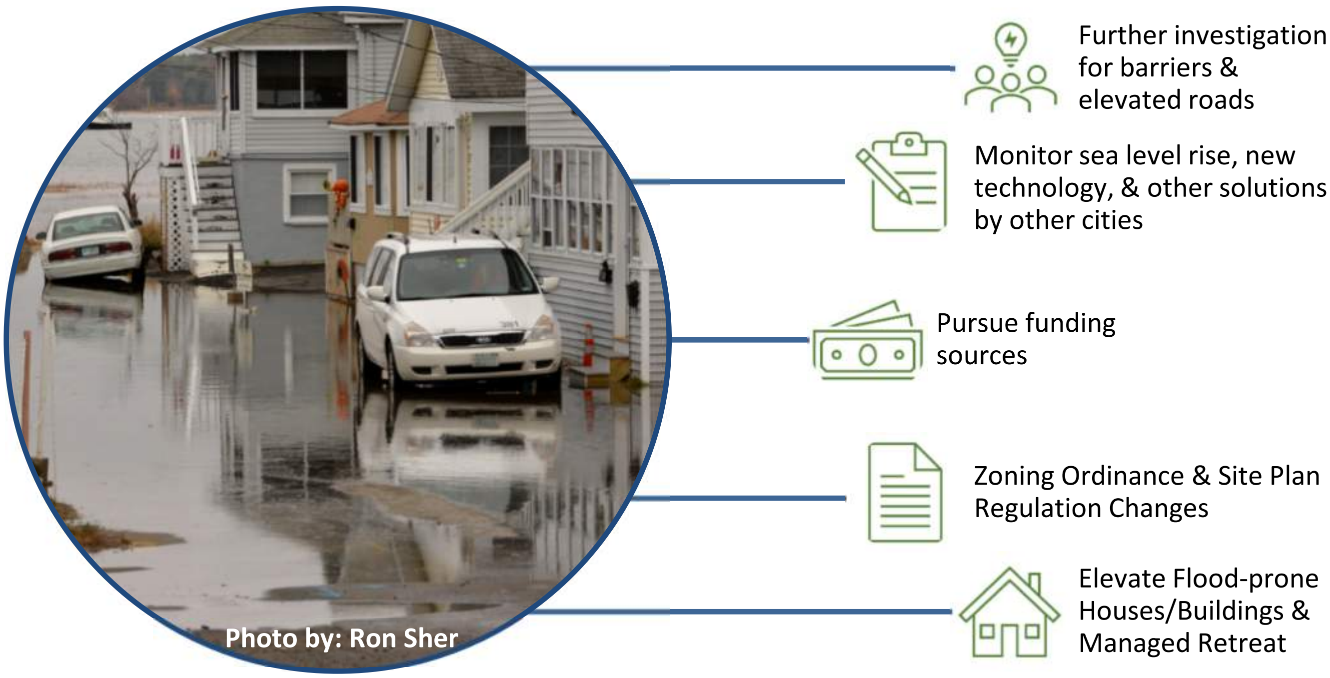

Figure 56 Summary of Alternatives

- **Continue to Pursue Funding**
  - Review existing State and Federal grant opportunities and pursue those that are applicable.
  - Monitor for new funding programs.
  - Evaluate the National Flood Insurance Program Community Rating System opportunities for potential eligibility into the program, which could provide discounts on flood insurance premiums (FEMA, 2021 Feb.).

- **Temporary Flood Barriers**
  - Investigate potential funding sources.
  - Decide what level of assistance, if any, the Town can provide to property owners in impacted areas for the purchase, deployment, and/or upkeep of temporary flood barriers. Options may include one or more of the following scenarios:
    - The Town owning and deploying temporary barriers at predetermined and feasible locations for protection of public property.
    - The Town subsidizing the purchase of temporary barriers (either with Town-only funding or through State or Federal grant programs obtained by the Town) for use to protect either public or private property as allowed by states and federal laws.
    - The Town supports privately owned temporary barriers and deployment in predetermined locations for successful implementation.
  - Launch a public outreach campaign about temporary flood barriers explaining the need for, and the short-term effectiveness, of this strategy, the level of assistance being offered

by the Town, suggested systems and where/how to obtain them, the Town's long-term plans for addressing flooding issues, etc.

- **Zoning Ordinance and Site Plan Regulation Changes**
  - Evaluate if there are existing residential structures within the flood zone that cannot be elevated above the BFE because of the Town's current Zoning Ordinance. Revise the regulations, as necessary and appropriate.
  - Evaluate whether rezoning flood-prone areas to limit expansion or reconstruction of improvements is a viable option.
  - Assess the effectiveness of the current Site Plan Regulations in managing expansion and growth in areas currently flood-prone.

- **Elevate Roadways**
  - Perform a detailed review of the private property, drainage system, and environmental impacts associated with elevating various sections of roadway.
  - Develop a phased approach to elevating roadways and construction of associated drainage improvements and related flood barriers with short, medium, and long-term actions:
    - Short-term
      - Roadways, or portions of roadways, that can be elevated without property acquisition or construction of extensive retaining walls.
    - Medium-term
      - Roadways that may require some property acquisition/modifications or retaining walls but can be implemented without significant public opposition or capital investment.
    - Long-term
      - Sections of roadways that would result in significant impacts to private property or the adjacent marshland if elevated, or that do not need to be elevated based on current (2020) flooding.
  - Proceed with engineering design and construction for the "Short-term" roadway sections, and begin planning efforts for "Medium-term" sections:
    - Topographical survey
    - Concept design (30% complete)
    - Initial design review with landowners
    - Final design
    - Final design review with landowners
    - Easement acquisition (as necessary)
    - Construction

- **Elevate Flood-prone Houses/Buildings and Managed Retreat**
  - Collaborate on planning process to explore building elevation and voluntary buyout projects in coordination with NHDES, regional planning commissions and the Town (recent proposal to NOAA to do this work that could begin in October 2021)
  - Hydraulic analysis of new flood storage areas to guide planning.
  - Support community engagement process to implement these alternatives.

- **Permanent Flood Barriers**
  - Continue coordination with local and federal permitting agencies to determine the level of permitting challenges a permanent barrier (either a wall or berm) would encounter.
  - If a permanent barrier is found to be feasible (environmentally and financially), proceed with engineering design and construction:
    - Topographical and drainage elements survey
    - Refined hydraulic analysis of the areas protected from flooding.
    - Concept design (30% complete)
    - Design review with landowners
    - Continued permit feasibility review with regulators.
    - Final design
    - Construction

Table 13
Hampton Harbor Flood Mitigation Summary

Category	Subcategory	Description	Objectives				Feasibility						Recommended to Further Investigate
			Reduce Flooding	Protect Private Property	Increase Public Safety	Minimize Environmental Impacts	Permitability	Constructability	Approximate Implementation Cost	Relative Operation & Maintenance Cost	Potential Short-Term Implementation	Potential Long-Term Implementation	
Resist with Infrastructure	Permanent Barriers	Install walls at low areas of Glade Path	✓	✓	~	~	X	✓	$20M - $40M	Medium	No	Yes	Yes
		Install walls at low areas of Island Path	✓	✓	~	~	X	~			No	Yes	Yes
		Install walls at low areas of Ashworth Ave	✓	✓	~	X	X	X			No	Yes	Yes
		Install walls at low areas of North of NH Route 101	✓	✓	~	~	X	~			No	Yes	Yes
		Vegetated earthen berms	✓	✓	~	X	X	X	$20M - $40M	Medium	No	Yes	Yes
	Temporary Barriers	Walls installed for duration of event and removed	✓	~	✓	✓	✓	~	$60 to $100 per linear foot barrier; $10,000 - $20,000 per house	High	Yes	No	Yes
	Elevate Flood-prone Roads	Glade Path	~	X	✓	~	~	~	$0.75M - $1.5M	Low	Yes	Yes	Yes
		Island Path	~	X	~	~	~	~	$1.5M - $3M	Low	Yes	Yes	Yes
		Ashworth Ave	~	X	X	✓	✓	X	$3.5M - $7M	Medium	No	No	No
Accommodate with Infrastructure	Existing Infrastructure	Elevate flood-prone houses/buildings	✓	~	~	✓	~	✓	$20K - $200K+ per structure	Medium	Yes	No	Yes
		Zoning & site plan regulation changes	✓	~	~	~	✓	✓	Minimal	Low	Yes	Yes	Yes
		Improve drainage	~	~	~	✓	✓	✓	Moderate*	Medium	Yes	Yes	No
		Tide gate improvements on stormwater outfalls	X	X	~	X	✓	✓	Moderate*	Medium	Yes	Yes	No
Retreat	Property Changes	Voluntary and assisted retreat/relocation	X	~	✓	✓	✓	✓	$500K per parcel; $20M for Glade $30M for Island $150M for Ashworth	Low	Yes	Yes	Yes

✓ = Good ~ = Fair X = Poor * Opinion of costs not developed for this alternative; anticipated to be less than $500K

Hoyle, Tanner & Associates, Inc.

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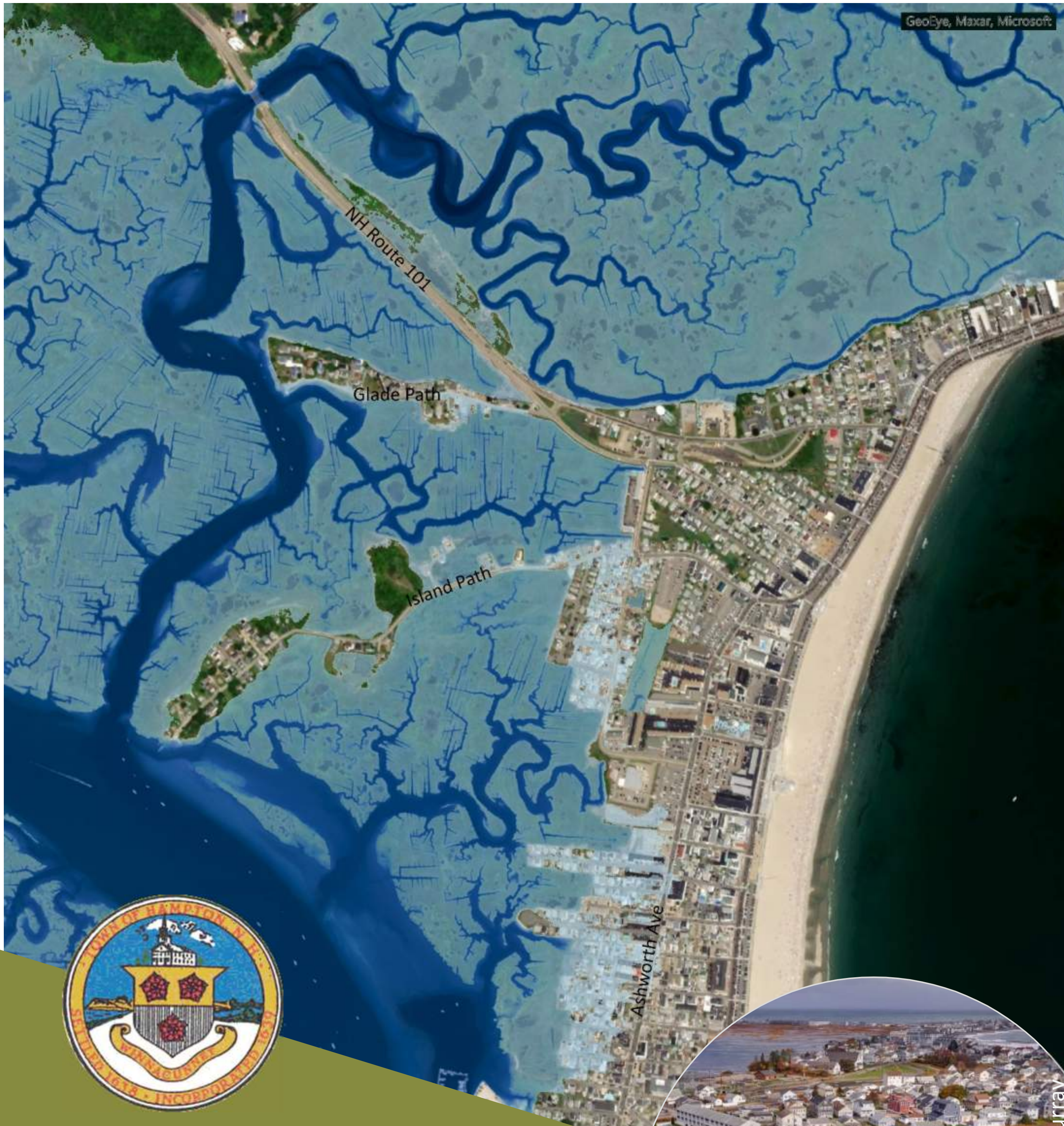
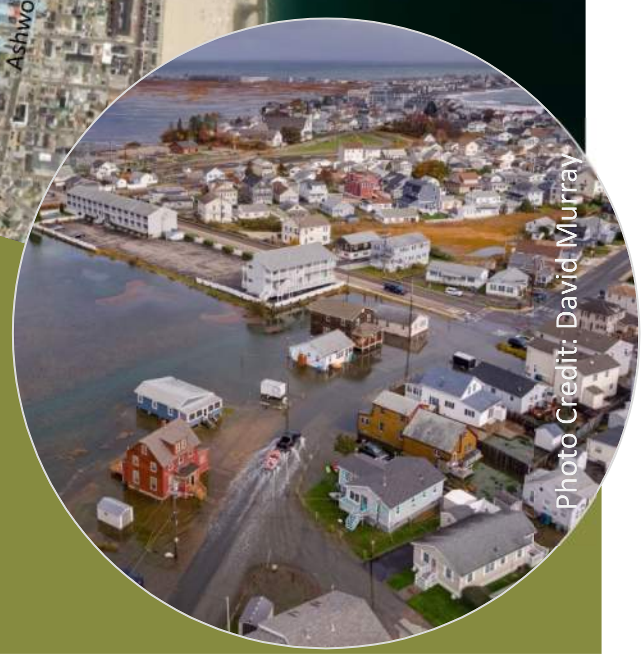




Hampton Harbor Flooding Evaluation – Appendices

March 31, 2021

Prepared by:

Hoyle, Tanner & Associates, Inc.

APPENDIX A

REPORTED FLOOD ISSUES AROUND HAMPTON

Engineering Report

March 2021

Questionnaire

Hampton: 12/11/2018 Public Informational Meeting

1. **What is your property address (House Number & Street)?**

2. **What is the use of your property?**
   a. Residential
   b. Commercial
   c. Other:

3. **How often have you had more than 6 inches of standing water on your property in the past 10 years?**
   a. Never
   b. 1 – 5 times
   c. More than 5 times

4. **What is the most amount of standing water on your property that you can remember?**
   a. None
   b. A few inches
   c. About a foot
   d. More than 1 foot
   e. More than 3 feet

5. **Does the water enter your building/home?**
   a. Yes
   b. No

6. **Do you have a basement with useable/finished living space?**
   a. Yes
   b. No

7. **Does the water force you to leave your property?**
   a. Yes
   b. No

8. **Does water prohibit you from leaving or getting to your property?**
   a. Yes
   b. No

9. **When did you first notice the flooding to be a problem?**
   a. In the past 5 years
   b. 5 – 10 years ago
   c. 10 – 20 years ago
   d. As far back as I can remember

10. **What concerns do you have?**

Hoyle, Tanner & Associates, Inc.

Questionnaire Results

Q1 What is your property address (House Number & Street)?

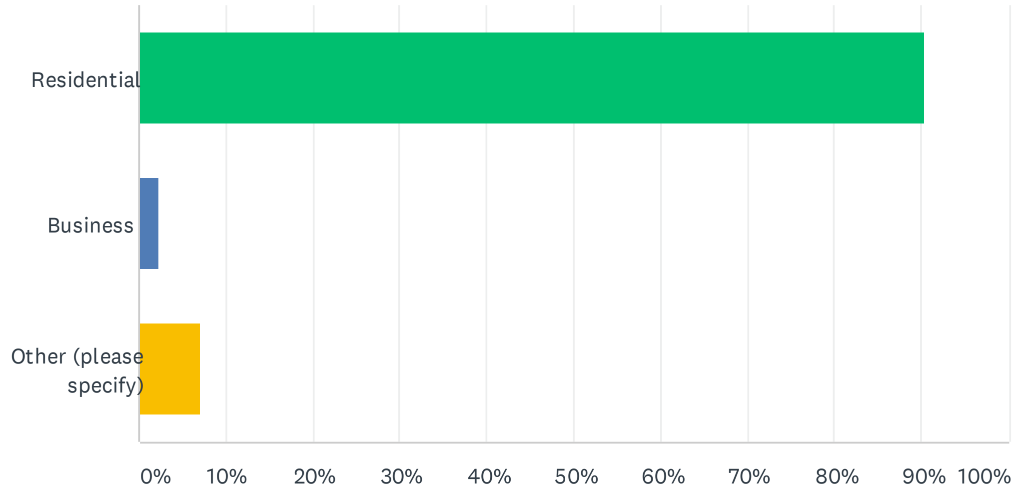
Answered: 42 Skipped: 0

#	RESPONSES	DATE
1	9 keefe avenue	5/7/2019 5:45 PM
2	7 Chase Street	4/16/2019 8:27 AM
3	20 fuller acres	4/8/2019 10:53 AM
4	16 Susan Lane	3/27/2019 7:27 PM
5	4 Keefe Ave	3/23/2019 10:58 AM
6	508 high st	2/14/2019 3:12 PM
7	11 bragg	2/11/2019 7:47 AM
8	11 F Street	1/26/2019 1:30 PM
9	110 Landing Road	1/24/2019 11:31 AM
10	18 Gill Street	1/3/2019 10:46 AM
11	16 Watsons Lane	1/1/2019 6:55 PM
12	42 Nudd Ave	12/23/2018 8:03 PM
13	11 Red Coat Lane	12/23/2018 3:57 PM
14	541 ocean blvd	12/22/2018 2:11 PM
15	4 meadow pond rd	12/20/2018 8:20 AM
16	8 Gentian Rd	12/19/2018 1:04 PM
17	22 Meadow Pond Rd. and 21 Meadow Pond Rd.	12/19/2018 11:43 AM
18	109/111 Kings Highway	12/18/2018 5:13 PM
19	59 Hobson Avenue	12/18/2018 2:02 PM
20	3 Witch Island Way	12/18/2018 7:39 AM
21	10 green st	12/17/2018 11:46 AM
22	3A Gentian Road	12/16/2018 4:56 PM
23	2 Gentian Road	12/15/2018 3:30 PM
24	35 Hobson Ave.	12/15/2018 3:21 PM
25	15-17 Machester St and 19-19r Machester St	12/15/2018 9:52 AM
26	21 Gentian Rd	12/15/2018 5:08 AM
27	16 Gentian	12/15/2018 5:04 AM
28	8 Greene St	12/14/2018 11:34 PM
29	44 Hobson Ave	12/14/2018 2:04 PM
30	75 Hobson Ave	12/12/2018 9:05 AM
31	15-17 Manchester St & 19-19R Manchester St	12/12/2018 9:04 AM
32	5 Manchester St	12/12/2018 9:00 AM
33	98 Ashworth Ave	12/12/2018 8:59 AM
34	7 Manchester Street	12/12/2018 8:58 AM
35	25 Jones Avenue	12/12/2018 8:57 AM
36	21-1 Riverview Terrace	12/12/2018 8:54 AM
37	511 Ocean Boulevard	12/12/2018 8:52 AM

38	4 Chase Street	12/12/2018 8:51 AM
39	60 Hobson Ave	12/12/2018 8:47 AM
40	5 Chase Street	12/12/2018 8:42 AM
41	22 Riverview Terrace Unit B	12/12/2018 8:34 AM
42	9 Manchester Street	12/12/2018 8:32 AM

Q2 What is the use of your property?

Answered: 42   Skipped: 0

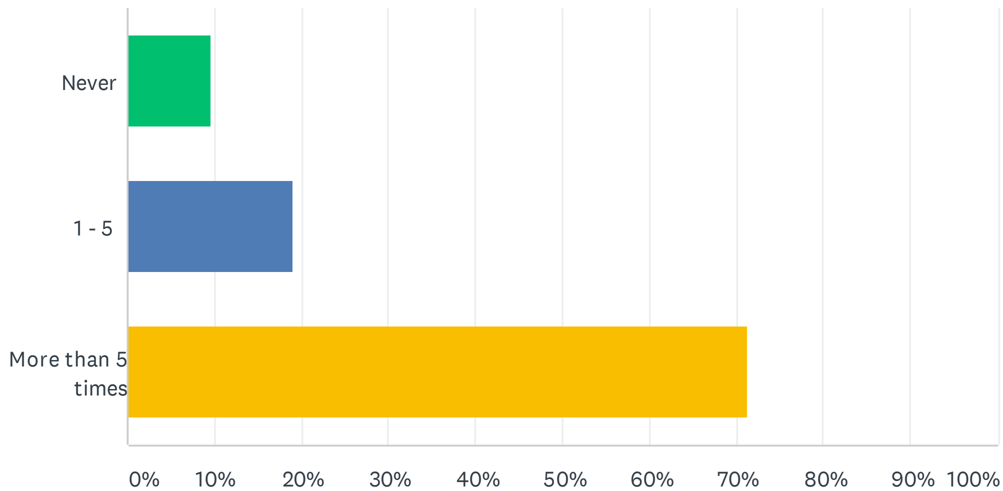


ANSWER CHOICES	RESPONSES	
Residential	90.48%	38
Business	2.38%	1
Other (please specify)	7.14%	3
TOTAL		42

#	OTHER (PLEASE SPECIFY)	DATE
1	Res. and Business	1/26/2019 1:30 PM
2	Summer cottage	12/17/2018 11:46 AM
3	business and pleasure	12/15/2018 9:52 AM

Q3 How often have you had more than 6 inches of standing water on your property in the past 10 years?

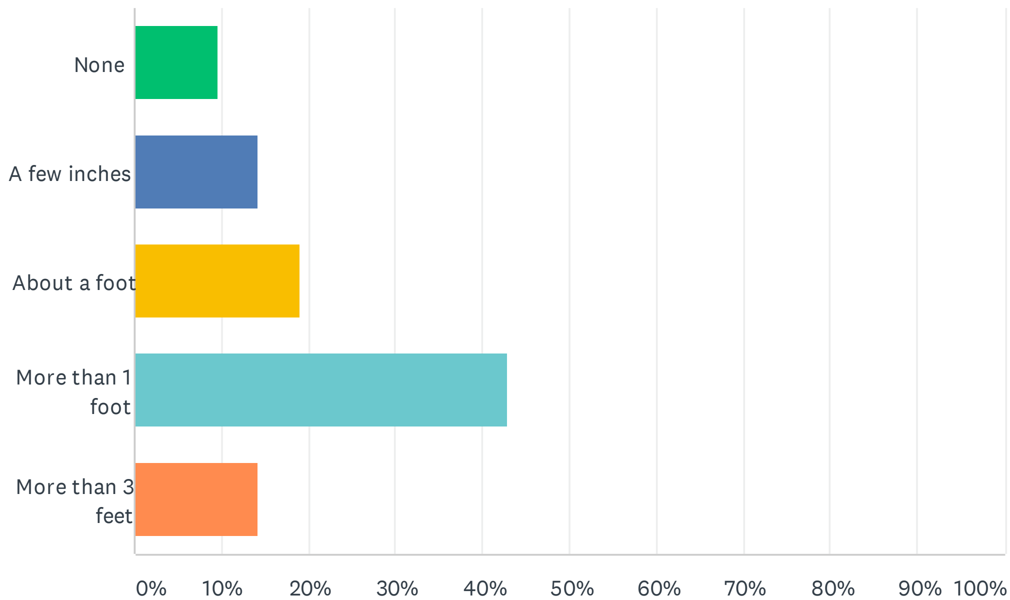
Answered: 42 Skipped: 0



ANSWER CHOICES	RESPONSES	
Never	9.52%	4
1 - 5	19.05%	8
More than 5 times	71.43%	30
TOTAL		42

Q4 What is the most amount of standing water on your property that you can remember?

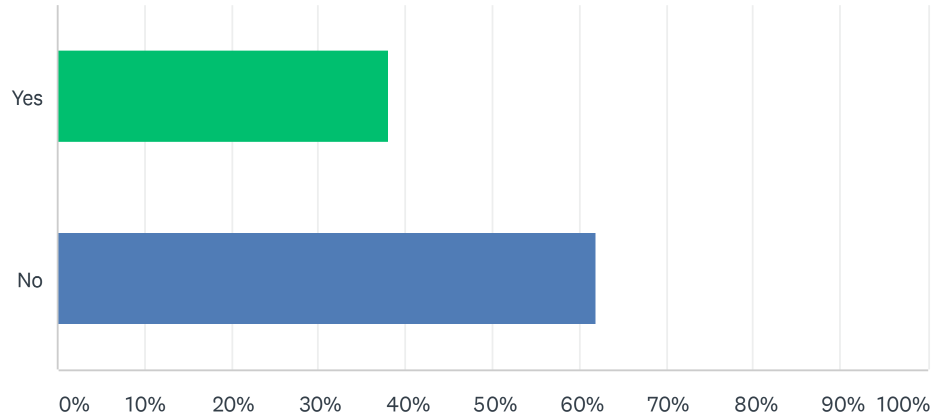
Answered: 42   Skipped: 0



ANSWER CHOICES	RESPONSES	
None	9.52%	4
A few inches	14.29%	6
About a foot	19.05%	8
More than 1 foot	42.86%	18
More than 3 feet	14.29%	6
TOTAL		42

Q5 Does the water enter your building/home?

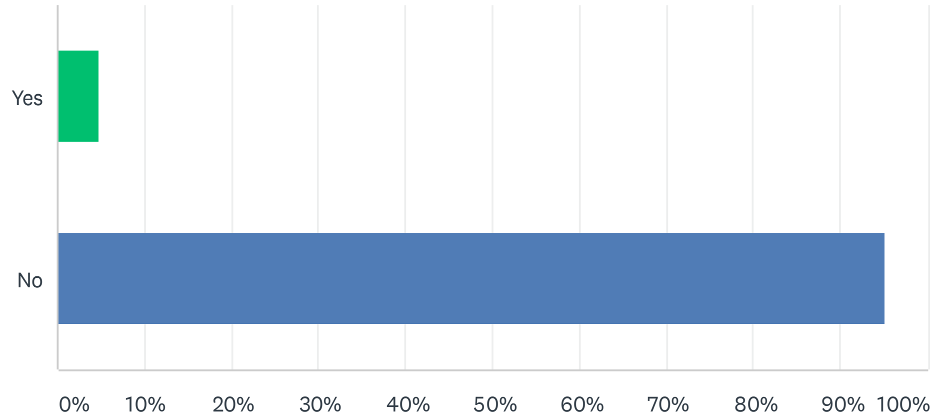
Answered: 42 Skipped: 0



ANSWER CHOICES	RESPONSES	
Yes	38.10%	16
No	61.90%	26
TOTAL		42

Q6 Do you have a basement with useable/finished living space?

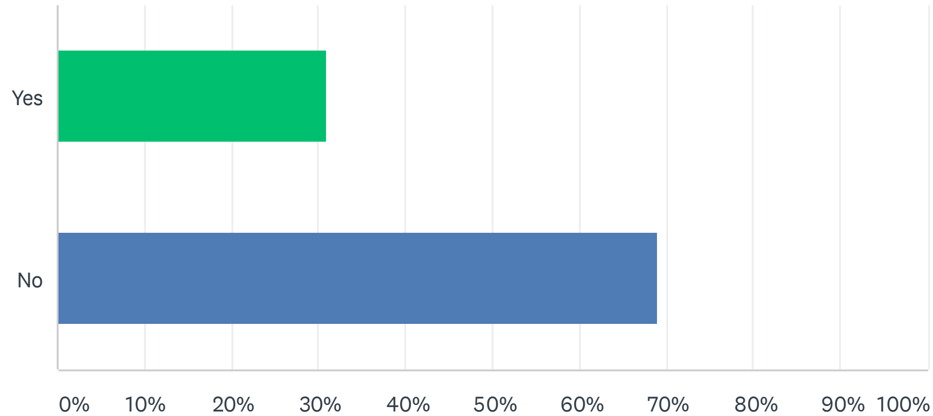
Answered: 42 Skipped: 0



ANSWER CHOICES	RESPONSES	
Yes	4.76%	2
No	95.24%	40
TOTAL		42

Q7 Does the water force you to leave your property?

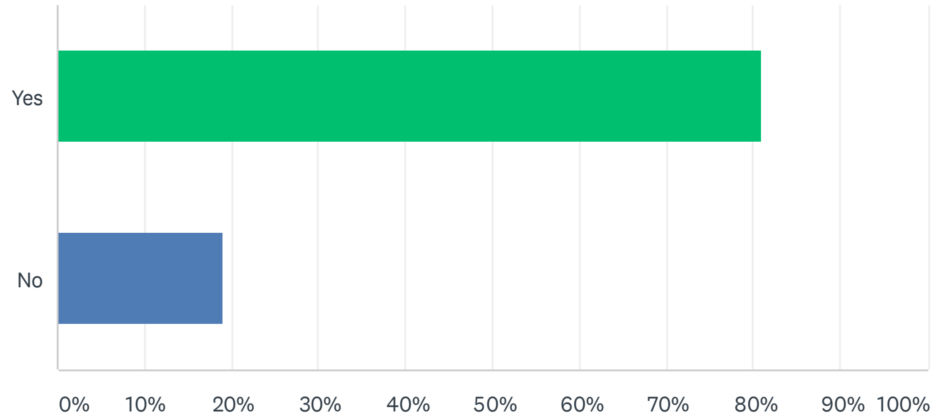
Answered: 42 Skipped: 0



ANSWER CHOICES	RESPONSES	
Yes	30.95%	13
No	69.05%	29
TOTAL		42

Q8 Does water prohibit you from leaving or getting to your property?

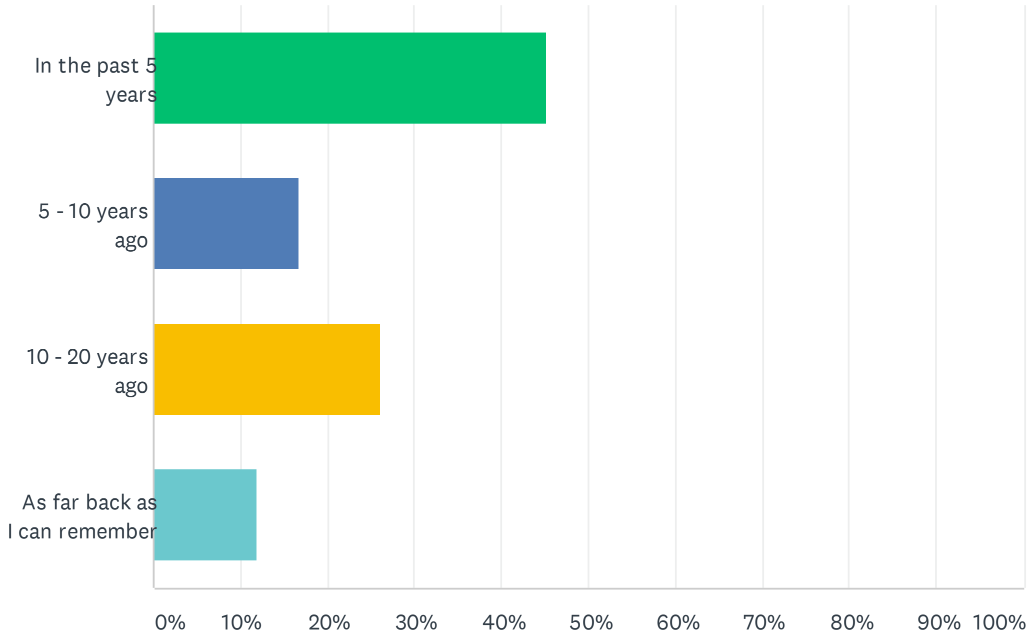
Answered: 42 Skipped: 0



ANSWER CHOICES	RESPONSES	
Yes	80.95%	34
No	19.05%	8
TOTAL		42

Q9 When did you first notice flooding to be a problem?

Answered: 42 Skipped: 0



ANSWER CHOICES	RESPONSES	
In the past 5 years	45.24%	19
5 - 10 years ago	16.67%	7
10 - 20 years ago	26.19%	11
As far back as I can remember	11.90%	5
TOTAL		42

Q10 What concerns do you have?

Answered: 36 Skipped: 6

#	RESPONSES	DATE
1	The most serious flooding I remember was the Blizzard of '78. There will always be some flooding during intense storms occurring at full or new moon or perigee. The beach continues to have its green spaces built/paved over, reducing absorbent surfaces. In addition, Hampton Harbor continues to fill with sand, displacing more water into the marsh and streets and stranding boats at low tide. The harbor dredging continues to be considered but not funded. Dredging the harbor is not a cure-all but would help. A barrier (@ two feet high) at the end of the streets off Ashworth Avenue, would also help (similar to the building on the south corner of Brown Avenue and Rt 101). Building houses on stilts will only reduce damage to the house, not any automobiles, and will reduce access to beach accommodations for disabled people, unless built with waterproof elevators to allow access to the house.	5/7/2019 5:45 PM
2	Rising flood water damaging property. Possible foundation/erosion settlement. Astronomical high tides in past 2 years have caused property damage. (Flood water running under house, damage to insalation, electrical service, settlement, vegatation damage.	4/16/2019 8:27 AM
3	The land continues to settle over the years but residents face obstacles when trying to fill in to compensate for this loss, I.e. special permits from the town and state. The town is also not willing to help divert the water in any way by building sea walls or perhaps dredging the existing marsh channels to make them deeper to contain more of the tidal water.	3/27/2019 7:27 PM
4	Why Keefe Ave was allowed to let the townhouses at the end of the street be in charge of the street and allowed not to complete the engineering design as promised to the town. This is a main reason this road now floods and water always pools about midway. The condo assoc for these townhouses made a promise and commitment to the town to maintain this road and they did not and do not in terms of flooding and potholes. AS you know they weren't even plowing it until this winter.	3/23/2019 10:58 AM
5	After asphalt /elevation change to high st rainwater cascades over new asphalt burm at end of driveway and floods yard.	2/14/2019 3:12 PM
6	Will we do anything so that the water doesn't come up from the Marsh	1/26/2019 1:30 PM
7	Our basement is on or near the grade. Water enters the sump and is pumped out when the ground saturates during prolonged heavy rain. No salt water has ever entered the building. Water prohibits us from leaving/returning to the property only during extreme tidal surge events and does not hinder our daily routine. Tidal flooding normally inundates culverts along our road during high tide events. I have noticed an increase in the level and frequency of salt water in drainage areas during normal high tide events over the last 5-10 years.	1/24/2019 11:31 AM
8	Water flows from the medical building lot next door onto Watsons Lane and into my driveway at 16. The swale put in by the town years ago near the mailboxes is inefficient. Need improved drainage for water exiting medical building lot onto Watsons Lane.	1/1/2019 6:55 PM
9	Large condo units/building on Wiinicunnett Road built behind property displacing water into our yard.	12/23/2018 3:57 PM
10	Tides affect our parking lot, flood our cars, basement with electrical equipment repairs every very high tide cycle	12/22/2018 2:11 PM
11	That no action will be taken	12/19/2018 1:04 PM
12	We need to fix the drainage system for one. End of Meadow Pond drain is always full - (22 End) at dead end. As well as the drain at Greene and Meadow Pond. The water from Meadow Pond is not the primary issue. Need to fix the streets. Have to park car on Kings Highway and wear fishing boots to get to house. Please work on repairing. Thank you for seeking input.	12/19/2018 11:43 AM
13	Mostly the flooding of Kings Highway	12/18/2018 5:13 PM
14	property and vehicle damage and cost for repairs	12/18/2018 2:02 PM
15	Flooding onHigh St & Kings Highway	12/18/2018 7:39 AM
16	Deterioration of the building's foundation	12/17/2018 11:46 AM
17	The flooding gets deep enough to have to park our car on other side streets because the roads to our house are flooded. This includes Green Street and Meadowbrook. When full moon, high	12/16/2018 4:56 PM

	tide and storm all coincide, the flooding creeps along our street towards the other side of the marsh. our street has marsh on 3 sides.	
18	Being trapped in the house by 4 or more feet of water. And being unable to drive out when the water on Greene st. exceeds 15 or so inches	12/15/2018 3:30 PM
19	My driveway floods and we need to move our vehicles. If the tides keep coming higher I believe that the water will get underneath our house and ruin the foundation. We have had our house raised once already.	12/15/2018 3:21 PM
20	1.worsening conditions 2. property destruction 3.decreasing property values 4. chronic monthly flooding that needs to be rectified FAST....not ten years from now.... 5. We have lost summer rentals..rents for my seasonal rentals and year round rentals are below market value but my property taxes have just increased over $1000 between both properties.....6. flood insurance is prohibitive....7..on question #5 water entered on jan 4th, 2018...I lost a boiler, hot water tank, refrigerator and flooring and heat and water in -7 degree weather.....	12/15/2018 9:52 AM
21	Damage to my car from frequently driving through knee high standing water on Gentian Rd. Inability to get to my property due to frequent flooding. Property erosion. Basement flooding	12/15/2018 5:08 AM
22	Subsidence. The entire area used to be a swamp but was filled in sometime after ~1935 going by USGS map I saw. How was that done? How much has it compacted since?	12/15/2018 5:04 AM
23	Flooding & drainage on Greene St. The existing drains are always full. The drain issue last year was on my property. Those leaking drains only add to the water saturation in the street & nearby properties.	12/14/2018 11:34 PM
24	Safety for emergency vehicles to get to our home if needed. The amount of trash and waste that floats around the home and from the marsh. Water and not being able to get to or from our home. Our property value decreasing due to the flooding concerns. The water/floods are getting more frequent and higher, last winter (2018(the water was a few inches from the sill and entering the house. We cannot park our cars in our driveways, they need to be moved prior to the flood, and at times, when you believe it won't be a flood, it is and you're scrambling to move your vehicle. We need to make sure that all personal items are tied down or they will float away and be lost.	12/14/2018 2:04 PM
25	Losing our property and water damage. Notes: Water entering building - no not yet.	12/12/2018 9:05 AM
26	Worsening!! Loss of Property Value (autos too)!! Getting NO HELP stopping water from coming up street. Notes: Property use - residential and rental. Water enter your building - it has. Water force you to leave - not usually.	12/12/2018 9:04 AM
27	The foundations are deteriorating	12/12/2018 8:59 AM
28	Damage to property	12/12/2018 8:58 AM
29	Snow plowed onto street drain in front of our driveway. Storm water run-off from the SANDS onto our property. Poor/no drainage on Blondeau's parking lot (Ashworth Ave) Notes: Does water enter the building: Yes crawl space.	12/12/2018 8:57 AM
30	With water rising more and more each year we need a flood wall.	12/12/2018 8:54 AM
31	Drainage on Ocean Boulevard (floods during storms) water backup from drains.	12/12/2018 8:52 AM
32	Tide gate at corner of Brown Ave & Highland does not always function due to debris getting stuck in doors preventing closure. Water overflows swale/creek. The swale/creek is overgrown that may be adding to the flood issue. Notes: Does flooding force you to leave - a couple of storms. Does it prohibit you from leaving - yes it can.	12/12/2018 8:51 AM
33	Want a wall like the one in Salisbury, MA. Want the Town to raise road. I want to be able to raise my driveway w/out ton of permits. Want to raise my property. Mooring Ave seems to flood (at least the street) less than Hobson Ave. You really need to talk to residents in Salisbury, MA where the wall built by the Army Corps of Engineers built. The people we spoke w/were extremely pleased w/the wall. Notes: Do 6 week rentals in summer. Water does not enter building but it is really close. Water prohibit you from leaving - Yes except when we bring down our raft - I have numerous pictures. 1st noticed the problem - I bought the house in 2015.	12/12/2018 8:47 AM
34	That we will end up with daily flooding and house damage. Flood last winter was higher than we've ever seen. We have had significant issues in recent years. Tide gate at Rt 101/Brown Ave - lock was broken for at least months which resulted in it not working properly.	12/12/2018 8:42 AM

	Additionally, trash/debris collects in that area which also impacts function of the gate. When this gate does not work properly the flooding of the area increases significantly - Lower Highland Ave, Church parking lot and Chase St. We have a swale that gets the river overflow. Also concerned of impacts of flooding on the integrity of our newer foundation (2003). Notes: Standing water on property - Roadway is 1-5 times, basement is >5). Water enters the home in the basement but not living space. Water force you to leave - No but water reached top of porch - did not lose power but tenants were told to evacuate if power was lost by Hampton PD. Water prohibit you from leaving - Several feet of water on street w/tidal surge after high tide). 1st notice the problem - Mother's Day flood 2005 and then nothing similar until multiple storms last winter. Our 4-5' basement filled w/water.	
35	Want to raise property but concerned about our cards and getting in and out of property.	12/12/2018 8:34 AM
36	Loosing my home due to damage and not having town of state fix it. Notes: We rent one cottage and live in the other. We move our cards (during flood events). Memorial weekend 2017 the worst and continues every month since then.	12/12/2018 8:32 AM


DEAD
END


Photo by: David Murray

Photo by: David Murray


Photo by: David Murray

Photo by: Ron Sher


18
MIVILLE
REMODELING, LLC
Commercial • Residental
582-4761
Photo by: Joel Albair

Photo by: Joel Albair

Date Saved: 6/15/2020
Document Path: U:\GIS Projects (Y)\4681-04\Maps\Flood Concern Areas_Portrait2.mxd

Flood Concern Areas

Hampton-Seabrook Estuary Flood Study

Flood Concern Areas Identified Through Public Input and Municipal Participation

(MMI, 2021)

0 0.05 0.1 0.2 0.3 0.4
MILES
1 in = 0 miles

N

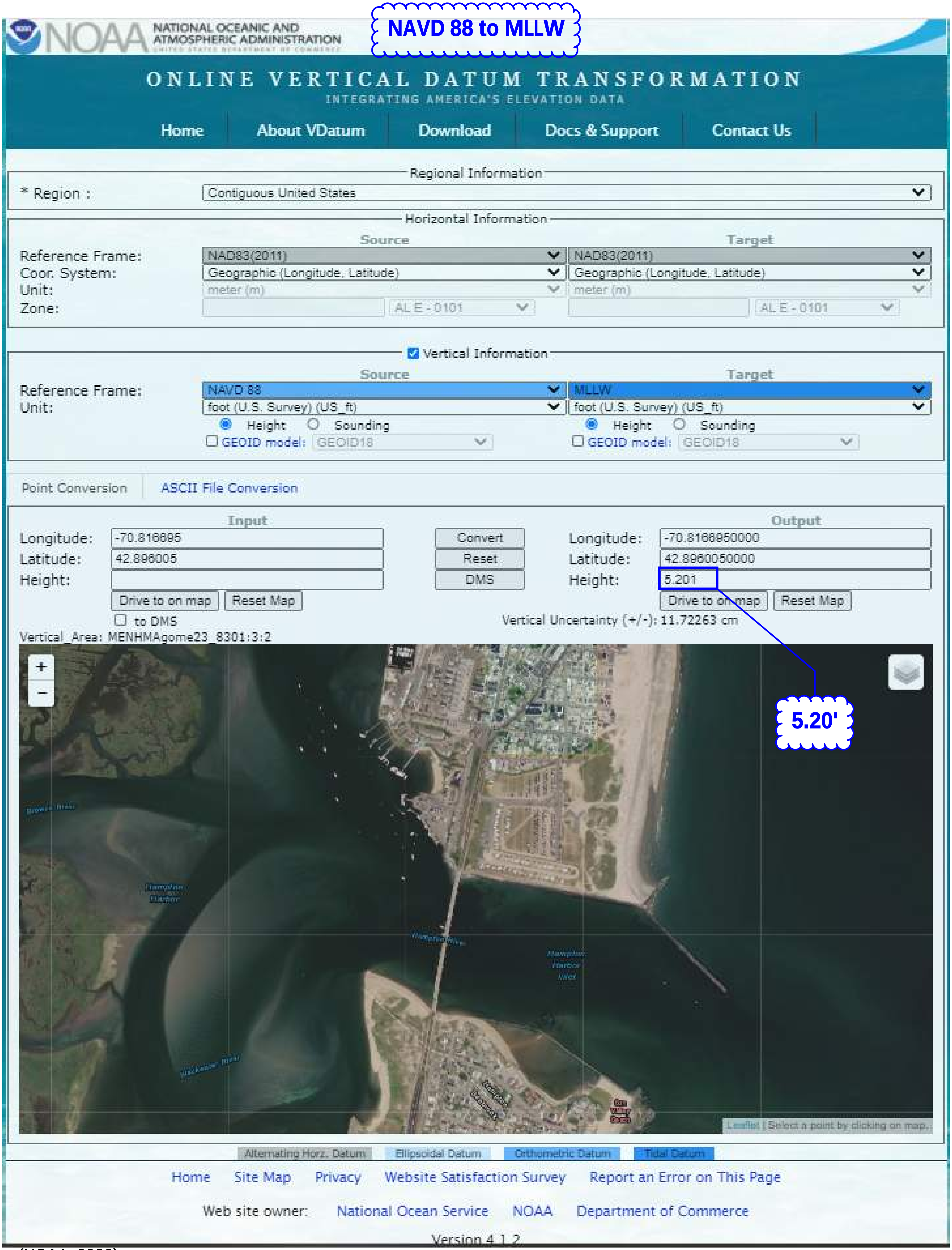
MILONE & MACBROOM
1 SOUTH MAIN STREET
WATERBURY, VT 05676
802.882.8335
WWW.MMINC.COM

APPENDIX B

NOAA TIDE DATUM INFORMATION

Engineering Report

March 2021


NAVD 88 to MLLW
NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION
ONLINE VERTICAL DATUM TRANSFORMATION
INTEGRATING AMERICA'S ELEVATION DATA
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Regional Information
* Region :
Contiguous United States
Horizontal Information
Source
Target
Reference Frame:
NAD83(2011)
NAD83(2011)
Coor. System:
Geographic (Longitude, Latitude)
Geographic (Longitude, Latitude)
Unit:
meter (m)
meter (m)
Zone:
AL E - 0101
AL E - 0101
Vertical Information
Source
Target
Reference Frame:
NAVD 88
MLLW
Unit:
foot (U.S. Survey) (US_ft)
foot (U.S. Survey) (US_ft)
Height
Sounding
Height
Sounding
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GEOID18
GEOID model:
GEOID18
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ASCII File Conversion
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DMS
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Drive to on map
Reset Map
to DMS
Drive to on map
Reset Map
Vertical Uncertainty (+/-): 11.72263 cm
Vertical_Area: MENHMAgome23_8301:3:2
5.20'
Leaflet | Select a point by clicking on map.
Alternating Horz. Datum
Ellipsoidal Datum
Orthometric Datum
Tidal Datum
Home
Site Map
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Website Satisfaction Survey
Report an Error on This Page
Web site owner:
National Ocean Service
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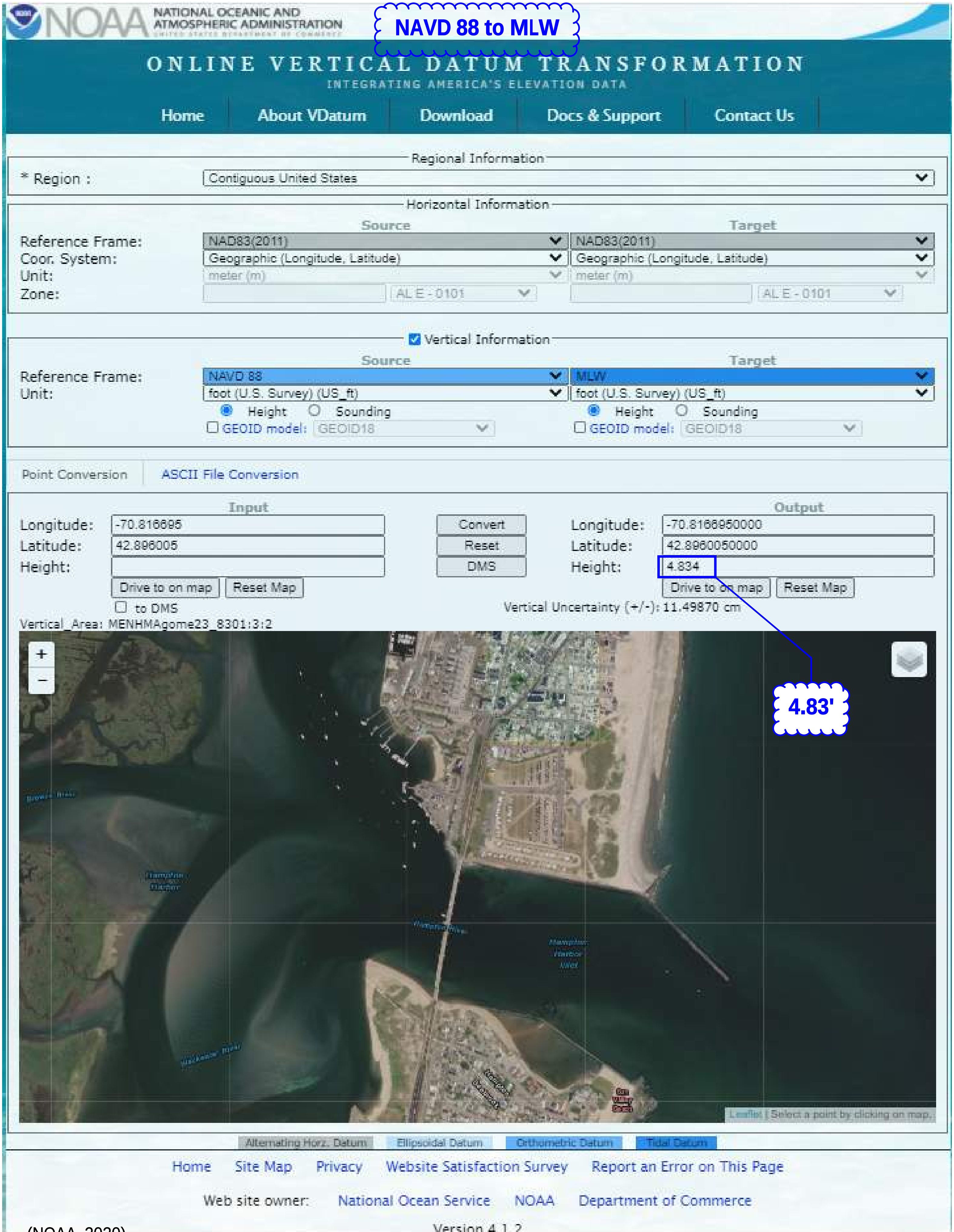
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* Region : Contiguous United States
Horizontal Information
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Target
Reference Frame: NAD83(2011) NAD83(2011)
Coor. System: Geographic (Longitude, Latitude) Geographic (Longitude, Latitude)
Unit: meter (m) meter (m)
Zone: AL E - 0101 AL E - 0101
Vertical Information
Source
Target
Reference Frame: NAVD 88 MLW
Unit: foot (U.S. Survey) (US_ft) foot (U.S. Survey) (US_ft)
Height Sounding
Height Sounding
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GEOID model: GEOID18
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Reset Map
to DMS
Convert
Reset
DMS
Output
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Latitude: 42.8960050000
Height: 4.834
Drive to on map
Reset Map
Vertical Uncertainty (+/-): 11.49870 cm
Vertical_Area: MENHMAgome23_8301:3:2
4.83'
Leaflet | Select a point by clicking on map.
Alternating Horz. Datum
Ellipsoidal Datum
Orthometric Datum
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Site Map
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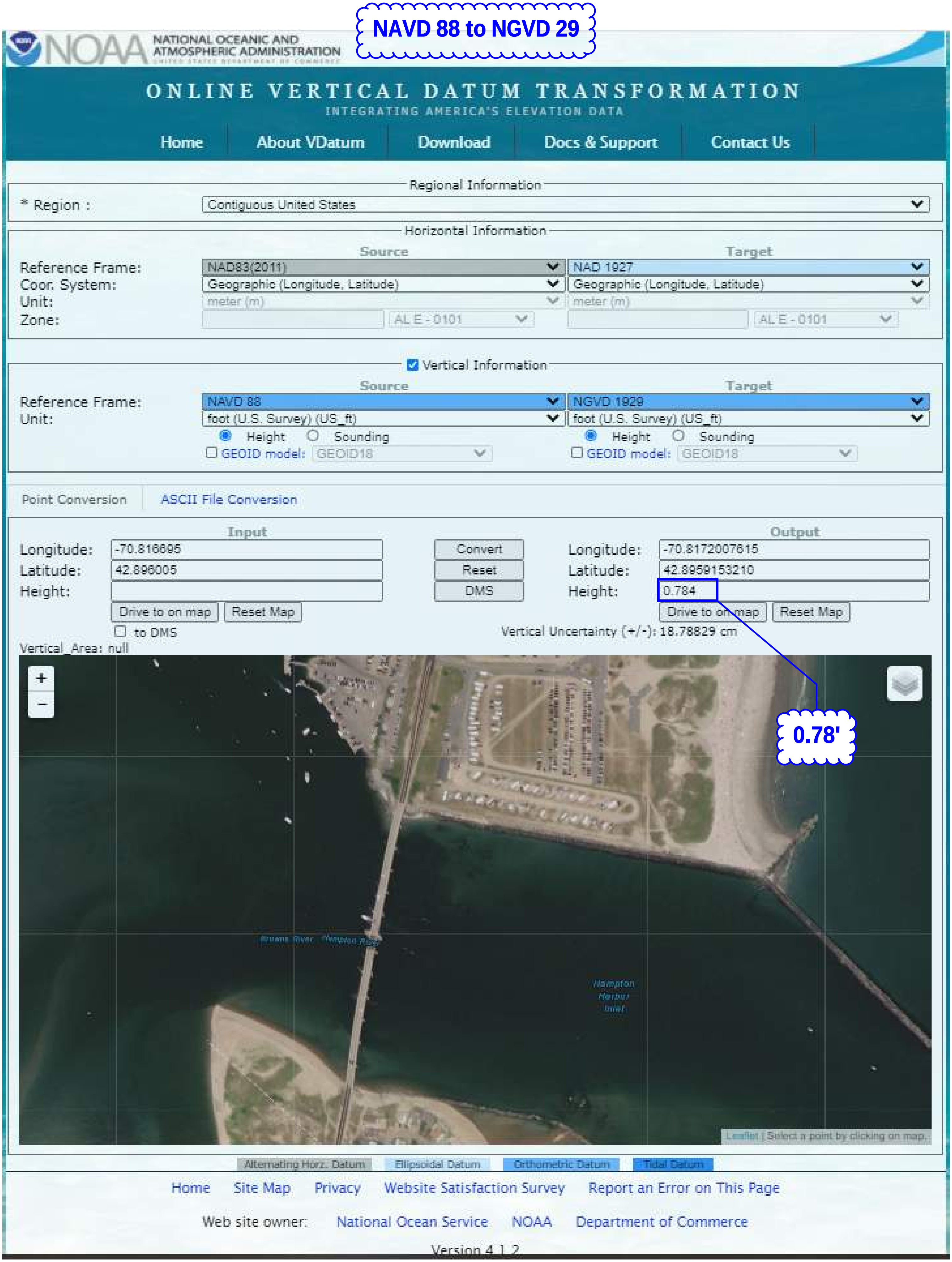
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Coor. System: Geographic (Longitude, Latitude) Geographic (Longitude, Latitude)
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Zone: AL E - 0101 AL E - 0101
Vertical Information
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Target
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Unit: foot (U.S. Survey) (US_ft) foot (U.S. Survey) (US_ft)
Height Sounding Height Sounding
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Point Conversion
ASCII File Conversion
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Latitude: 42.896005
Height:
Convert
Reset
DMS
Output
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Drive to on map
Reset Map
to DMS
Vertical Uncertainty (+/-): 18.78829 cm
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Hampton Harbor Inlet
Leaflet | Select a point by clicking on map.
Alternating Horz. Datum
Ellipsoidal Datum
Orthometric Datum
Tidal Datum
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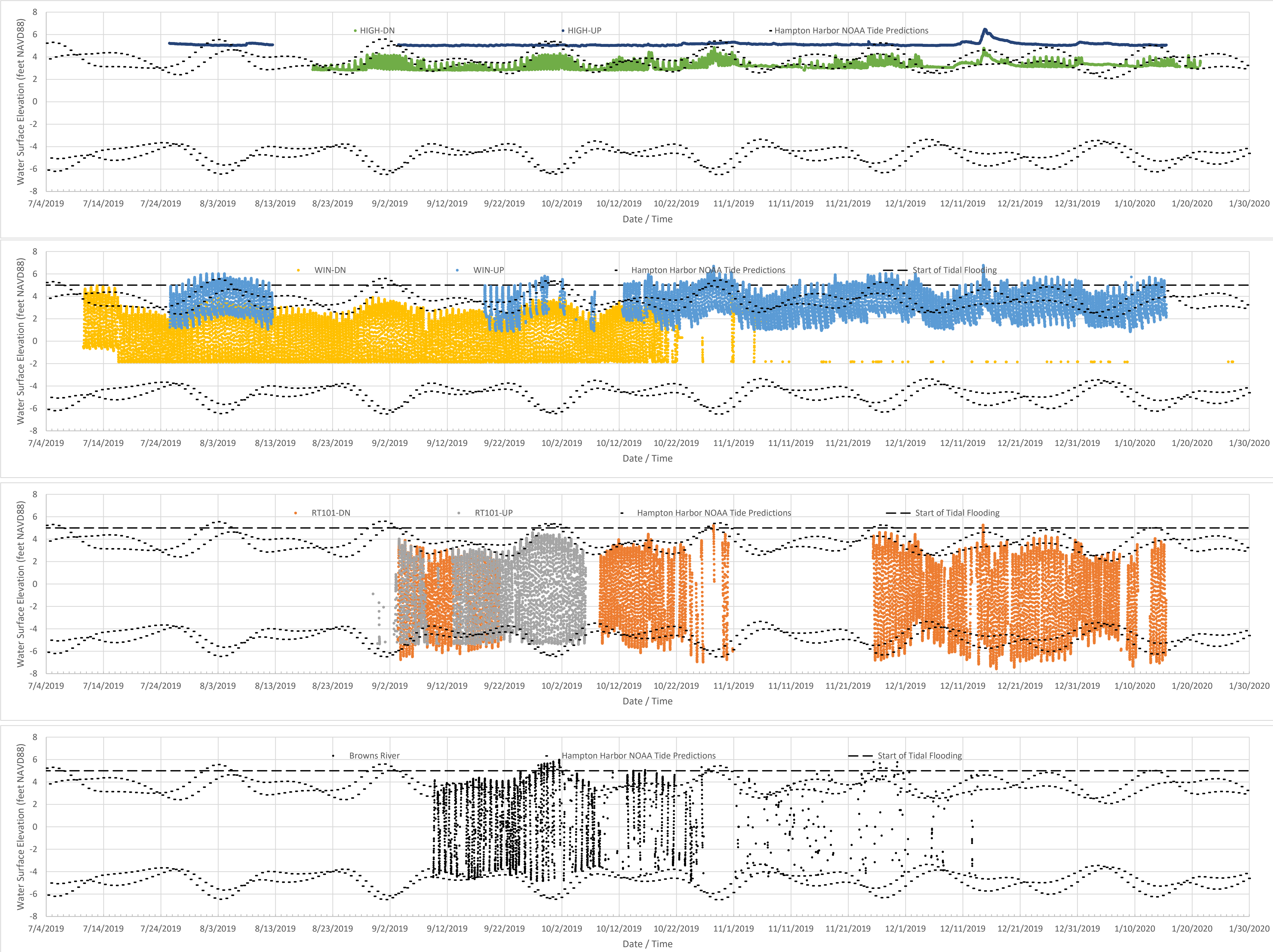
APPENDIX C

WATER LEVEL MONITORING

Engineering Report

March 2021

Hoyle, Tanner & Associates, Inc.



HIGH-DN
HIGH-UP
Hampton Harbor NOAA Tide Predictions
Water Surface Elevation (feet NAVD88)
Date / Time
WIN-DN
WIN-UP
Hampton Harbor NOAA Tide Predictions
Start of Tidal Flooding
Water Surface Elevation (feet NAVD88)
Date / Time
RT101-DN
RT101-UP
Hampton Harbor NOAA Tide Predictions
Start of Tidal Flooding
Water Surface Elevation (feet NAVD88)
Date / Time
Browns River
Hampton Harbor NOAA Tide Predictions
Start of Tidal Flooding
Water Surface Elevation (feet NAVD88)
Date / Time
8 6 4 2 0 -2 -4 -6 -8
7/4/2019 7/14/2019 7/24/2019 8/3/2019 8/13/2019 8/23/2019 9/2/2019 9/12/2019 9/22/2019 10/2/2019 10/12/2019 10/22/2019 11/1/2019 11/11/2019 11/21/2019 12/1/2019 12/11/2019 12/21/2019 12/31/2019 1/10/2020 1/20/2020 1/30/2020

(MMI, 2021)

Meadow Pond Tide Monitoring

Data Notes

Prepared by – Gopal Mulukutla

Updated -February 10th 2020

Introduction and Summary

Site Attributes

This document summarizes QA/QC process and other data processing steps implemented to develop a water level, salinity/conductivity and temperature dataset for Meadow Pond locations (described in Table 1 and Table 1)

Table 1: Site Attributes

Site #	Descriptio	Site code	Lat	Long	Installed	Notes
1	High Street	HIGH-UP	42.941611	-70.800656	Summer 2019	upstream
2	High Street	HIGH-DN	42.941425	70.800501	Summer 2019	downstream
3	Off Dunvegan Woods Rd.	DVG	42.936485, -70.806108		Summer 2019	
4	Winnacunnet Rd.	WIN_UP	42.927472	-70.800595	Summer 2019	upstream
5	Winnacunnet Rd.	WIN_DN	42.927274	-70.800892	Summer 2019	downstream
6	RT-101	RT101_UP	42.92214	-70.823229	Summer 2019	upstream
7	RT-101	RT101_DN	42.922056, -70.823391		Summer 2019	downstream
8	Brown's River at Glade Path	BROWN	42.916578	-70.823403	Summer 2019	

Table 2: Elevation of tide monitoring station per NAVD88 datum

Station Code	Doucet Pt #	UTM Y (US Feet)	UTM X (US Feet)	Elevation (NAVD88 ft)	Source
HIGH_UP	6024	162129.49	1216138.4	7.0038	Doucet
HIGH_DN	6008	162070.99	1216182.4	4.3509	Doucet
DVG	7000	160275.42	1214753.4	4.337	Doucet
WIN_UP	7010	156979.38	1216220	2.63	Doucet
WIN_DN	7012	156915.59	1216136.3	-0.02	Doucet
RT101_UP	7015	154948.73	1210230.8	-3.56	Doucet
RT101_DN				-8.025	Estimated from RT-101 UP
BROWN		152952.4	1210142.3	-3.99	Extracted from DEM- 2013 USACE NAE Topobathy Lidar

Summary

Data was compiled in 10-minute increments. Each dataset has its own start and end dates, but each data row is data for a 10-minute timestamp. The data sets can be aligned easily in Excel. To examine data please see accompanying CSV files supplied with this document. Each station's data is compiled as one file **<STATION_NAME_start-date_to_end-date.CSV>.** Variable names in the headers are self-explanatory and described below.

Table 3: Variable names and description

Variable Name	Description	Units/Format
Time_EST	Time stamps per EST	yyyy-mm-dd hh:mm
Depth_ft	Water depth uncorrected	Depth as measured Feet
Depth_ft_NAVD88	NAVD88 Benchmarked water depth	NAVD88 Feet
Cond_mScm	Conductance	Milli Siemens per centimeter (or dS/m)
Temp_C	Temperature	Celsius
Salinity_PSU	Salinity	Practical Salinity Units (PSU)

A summary plot of all elevations is compiled in the plot below (Figure 1).

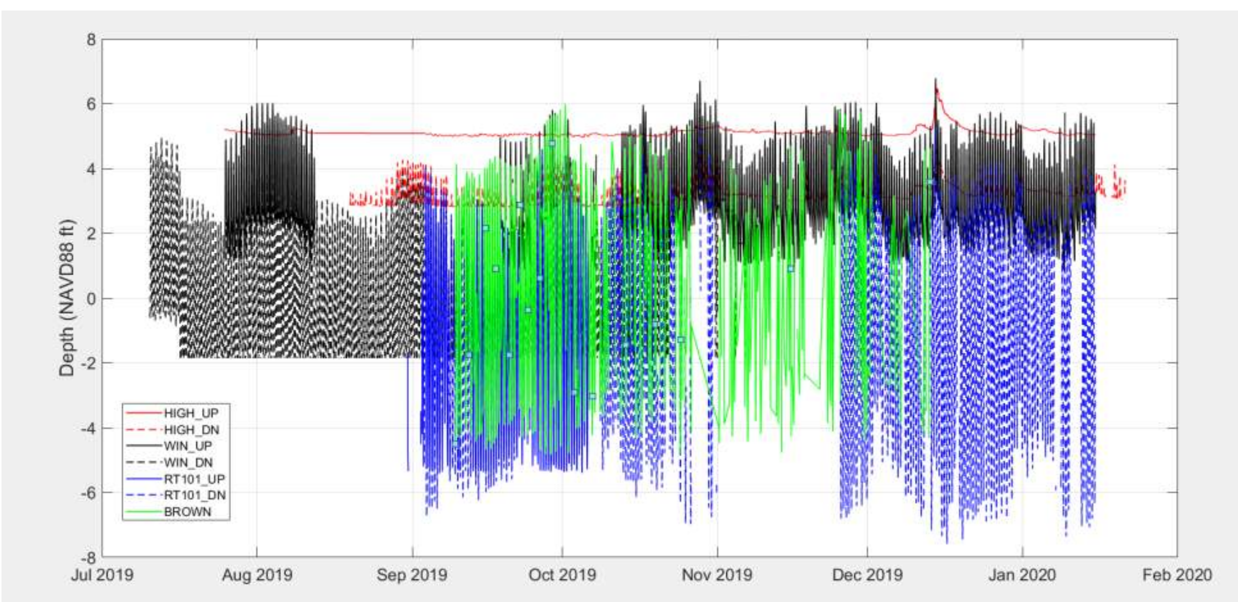


Figure 1: Tidal Elevations for 7 stations in the Meadow Pond and a connected waterway (Brown's River)

Notes on Data Completeness and Caveats

1. **Installation:** Stations were mostly installed in July -September 2019.
2. **Accuracy of Time: Accuracy of each time stamp is +/- 5 minutes:** Each node was equipped with a real time clock. This particular component's time was known to drift about 8 minutes for every 150 days of deployment. This was checked by noting the time of data collection (either by direct means or wireless data collection with a clock of its own). Each timestamp was then corrected for this drift before compiling the dataset. While there is every reason to believe that timestamps are accurate to less than 1 minute, as a conservative measure it was decided to give a +/- 5 minute value. Time reported in the dataset is only Eastern Standard Time (with no change coming from Daylight Saving Time)
3. **Data Caveat 1:** One station (WIN_UP) was vandalized within couple of weeks of deployment. At the same time, the tube in which the sensor was installed, got pried loose. The tube and the node electronics were installed at a nearby location. Elevation per datum (NAVD88) was collected only at the new station. The initial segment of data (collected before station was vandalized) was depth-corrected so that it aligns with the rest of the data.
4. **Data Caveat 2: One sensor (WIN_DN)** experienced a disruption from bank erosion that caused the node's depth sensor to malfunction. Starting with high values of bad data in October.
5. **Data Caveat 3: One sensor (RT101_UP)** also had malfunctioning electronics. Due to the location of installed electronics box being shaded to the south, there was condensing moisture and sensor itself started to freeze over during low tide measurements. This caused a series of issues that were not easy to resolve.
6. **Data Caveat 4: One sensor (Dunvegan Woods, DVG)** was installed at a location only accessible via canoe. This sensor and its data is not been accessible since it was installed.
7. **Data Caveat 5: One sensor's data (Brown's River, BROWN) could not be accessed fully.** The sensor was equipped with a wireless device to relay data to DPW. Due to an antenna malfunction, data is only partially received. Due to the

inaccessible location of the sensor electronics, complete data will be downloaded, and dataset revised in February-March 2020.

8. **Data Caveat 6:** Benchmark depths for two locations (BROWN and RT101_DN) could not be performed by Doucet Survey. (a) BROWN the tube was not installed on time, and (b) RT101_DN was installed at a point that was too deep to locate. For BROWN, the benchmark elevation of the point was extracted from a USACE Lidar DEM downloaded from NOAA Digital Coasts. For RT101_DN – a comparison of depth data with benchmarked depths from the neighboring RT101_UP station showed that there is a high degree of correlation between these data (see figure 2, below). Assuming that the channel is not constrained for flow, the benchmarked depth of the tube at RT101_DN was extracted, and used to develop the dataset.

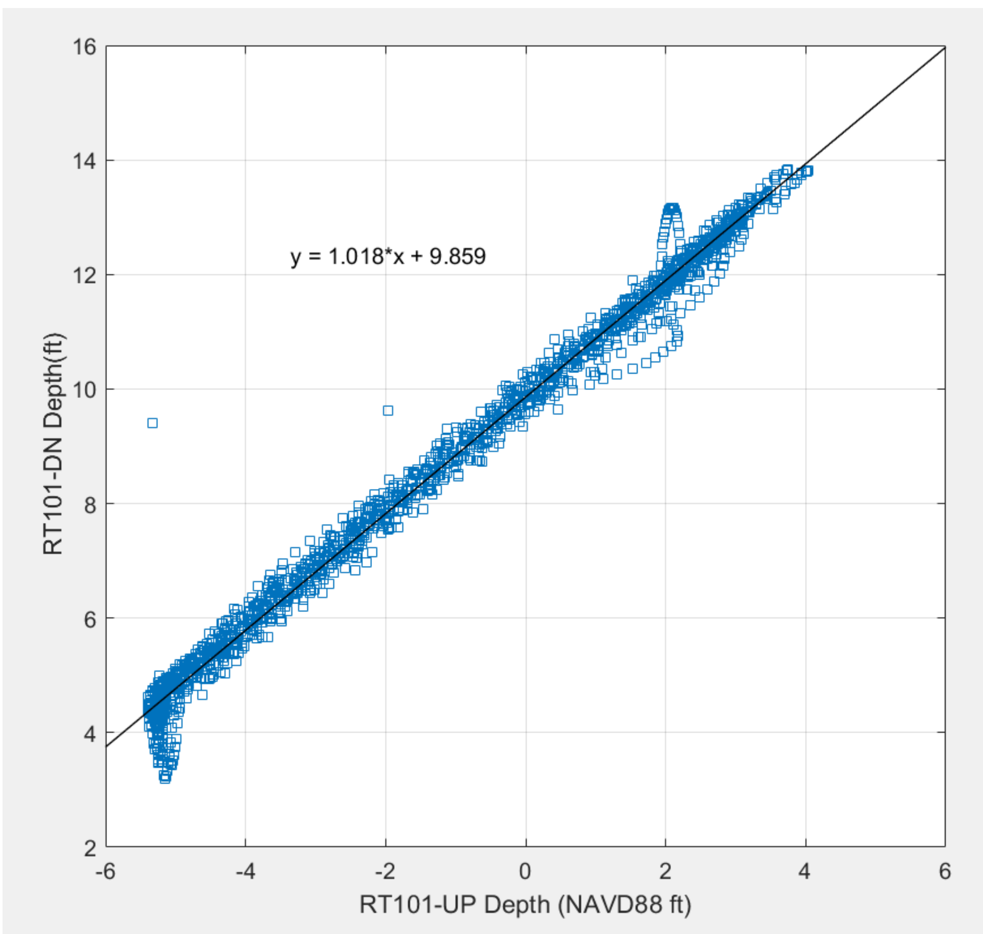


Figure 2: Comparison of water depths at RT101_DN with concurrent benchmarked depths at RT101_UP

9. **Data Caveat 7: One road crossing (Winncunnet Road,WIN_UP and WIN_DN) shows excessively high salinity data.** Sensors at both of these stations show high salinity. The causes for this is not fully known at this time.

Notes on Data Processing

Raw data was collected roughly every 5 to 8 minutes. A new energy saving device was used to power the electronics using a nano-timer (TI electronics). The timer itself turns on based on a complex algorithm dictated by an assigned resistor. Due to changing temperature and environmental conditions the resistor's value can change thus varying the timing of power-up and sampling times. Data collected this way have non-uniform timestamps. Following are data processing steps implemented to develop the datasets.

Step 1: Remove outliers: Initially raw data was examined for outliers and bad data, and this data was removed.

Step 2: Correction of clock drift: There is drift in the clock of approximately 1 hour for every 150 days of deployment. This was corrected in the time stamps.

Step 3: Transformation of Local Time to EST: Deployment spans EST and EDT. The EDT timestamps were transformed so the entire data has timestamps in EST.

Step 4: Alignment of data to uniform 10-minute timestamps: Each data point was aligned to the nearest 10-minute timestamp. A window 7.5 minutes was used to search for data, and if no data was present, a blank line (NaN- Not a Number) was added to the data.

Step 5: Gap filling: A very narrow gap filling procedure. Gap filling was performed on missing sample lengths of less than 2 hours. Missing data points were linearly interpolated with neighboring data.

Step 5: Water levels benchmarked to datum: Datum supplied by Doucet Survey (or extracted by other means), for each station was used to develop benchmarked water level. Figure below describes the assumptions made in the calculations.

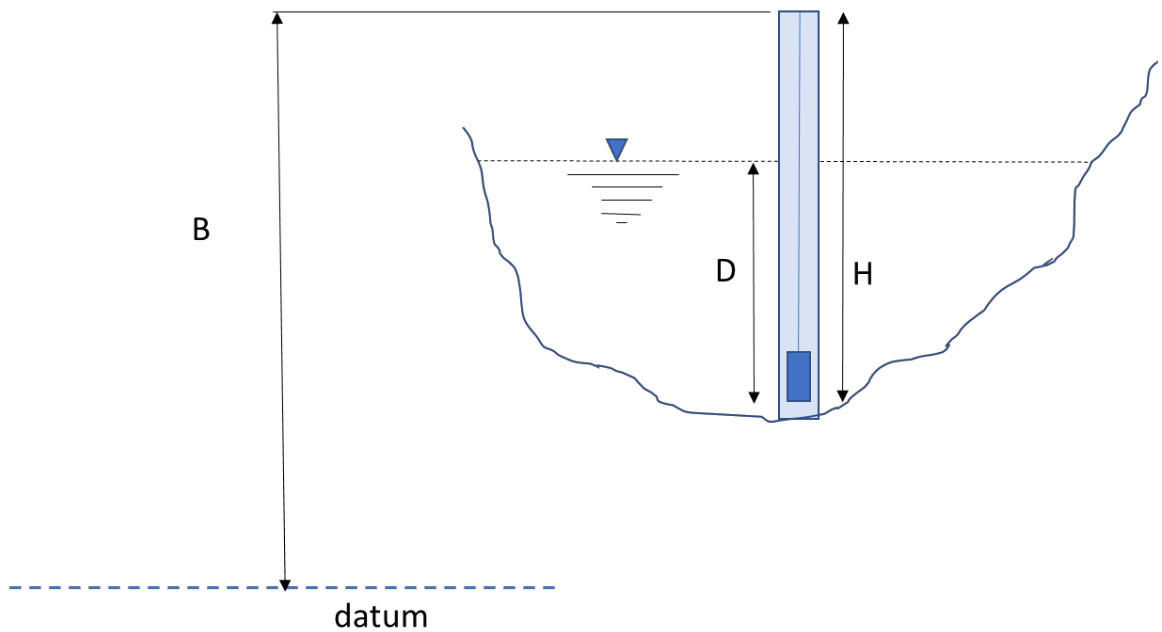


Figure 3: Schematic describing the relationship between installed sensor, and benchmark.

B= benchmarked depth of top of PVC tube (surveyed by Doucet)

D= D(t), instantaneous water depth measured by sensor.

H= Depth of sensing element from top of PVC tube

D_b = benchmarked water depth

Benchmarked water depth is given by:

D_b (t)= B-H+D (t)

Step 6: Estimate of Salinity: Conductivity and temperature data was used to estimate instantaneous salinity. A method described in Fofonoff, and Millard. (1983) was used to calculate salinity.

Some Data Highlights (High Street intersections) (HIGH-UP, and HIGH-DN)

Sensor nodes were installed at these stations in late summer. There was delay of a few weeks between the installation of the upstream and downstream stations.

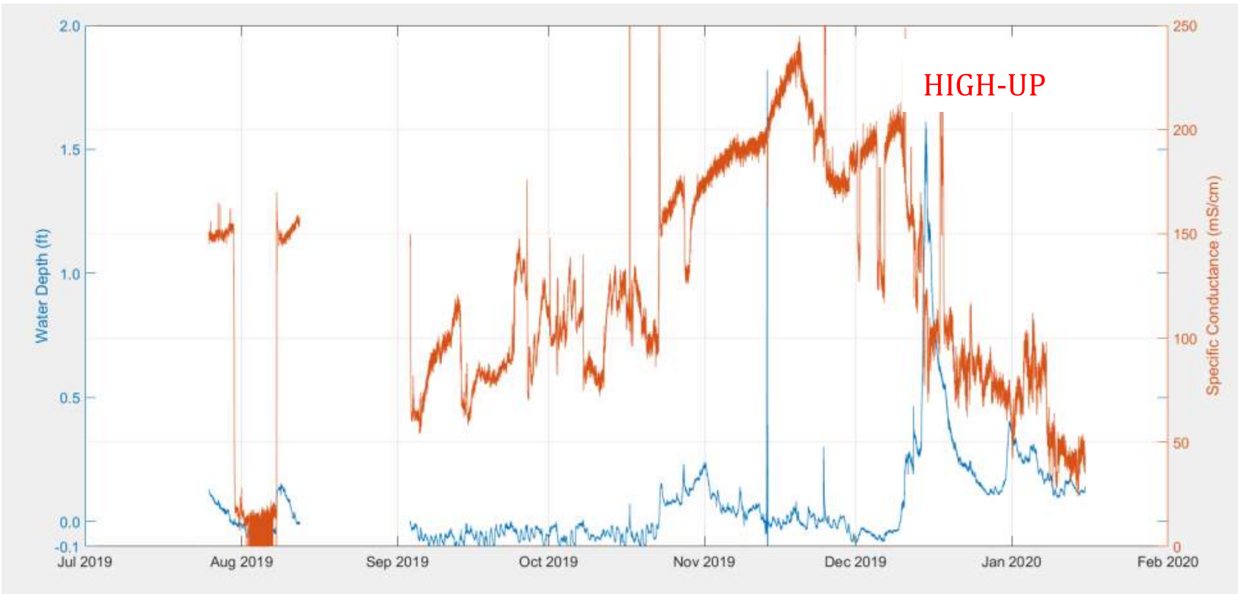


Figure 4: Unprocessed raw data of water level and conductivity at HIGH-UP. with non-uniform timestamps. Late summer data at this upstream station is sparse, as it was taken down due to low flows. The stream is marked by long periods of shallow water depth, very little tidal activity and short but sustained flashy high flows during precipitation events. This location shows a large spike in water level in mid-December, suggesting a snowmelt or rain event.

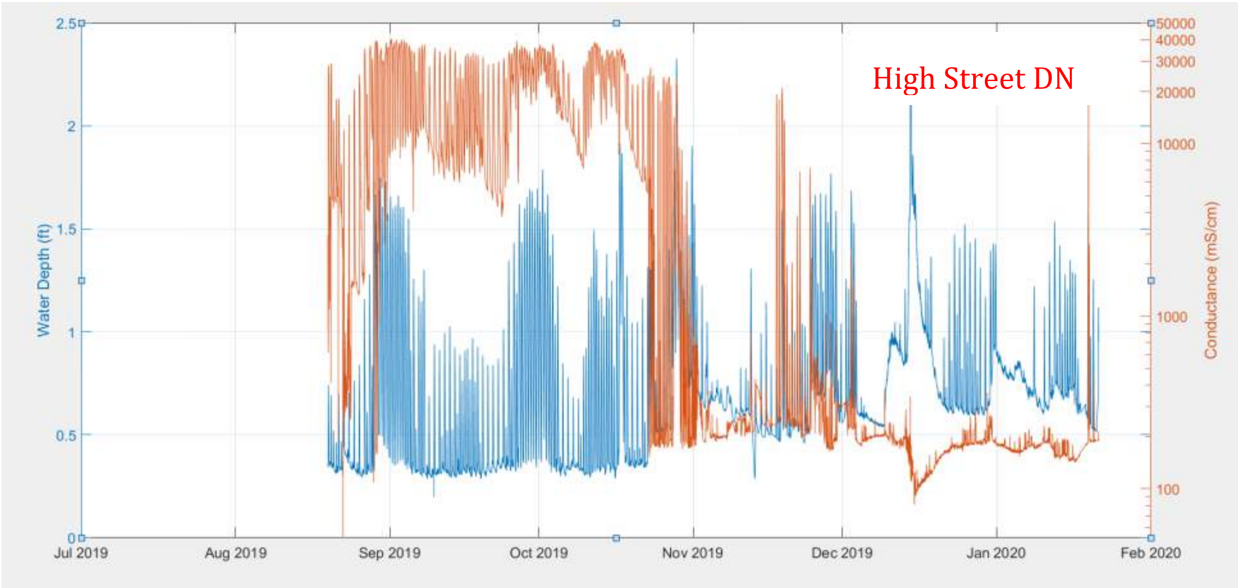


Figure 5: Unprocessed (collected as-is) data is shown for High Street-downstream station. Conductivity (right) axis is in log-format, to show the transition from saltwater dominant to freshwater dominant system as the season changes from summer to fall. Note that there is a drift in the clock of about an hour that is not corrected here. The high flow even in mid-December coincides with a large precipitation event.

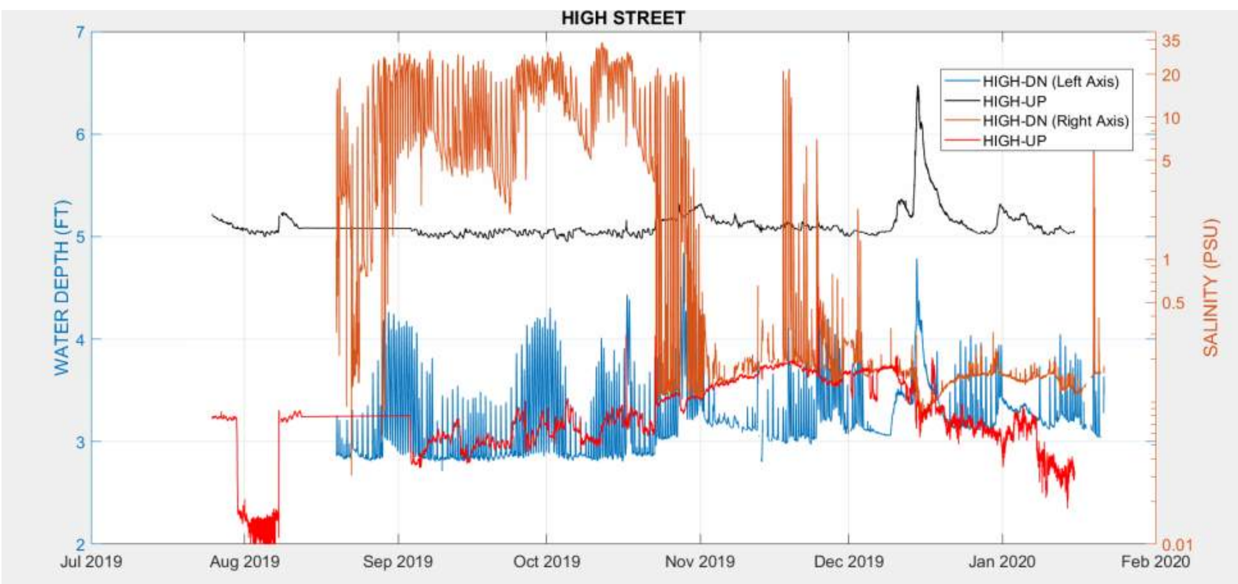


Figure 6: A compilation plot of data from both the stations. Left axis is benchmarked water depth, and right axis is salinity.

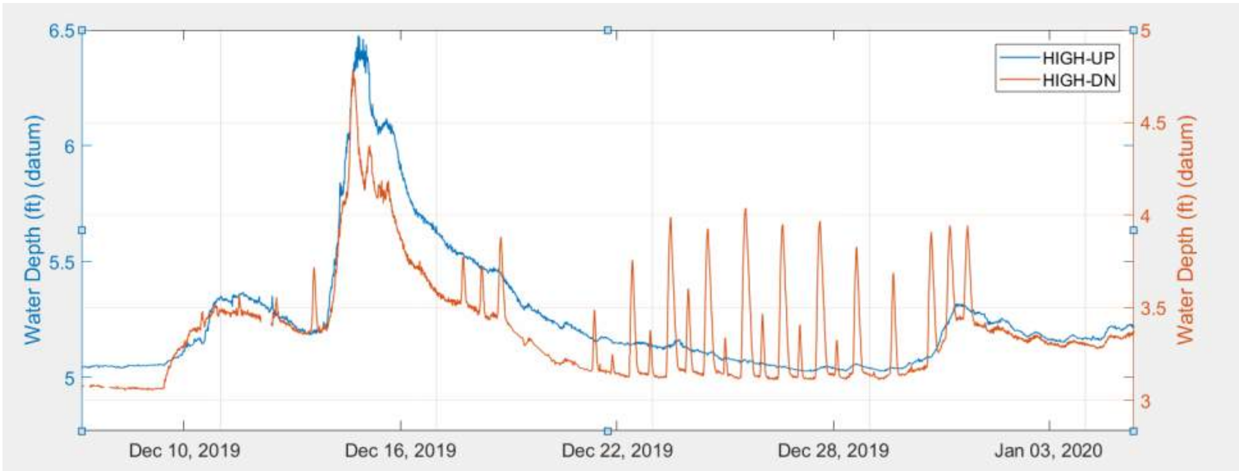


Figure5: A closeup of mid-December storm event shown for processed, benchmarked water depth data. Both left and right axes are water depth, plotted separately to align patterns. Downstream location shows some tidal activity, but high flows mark low salinity (not seen here). UNH weather station recorded 75.2 mm of rainfall during this time period.

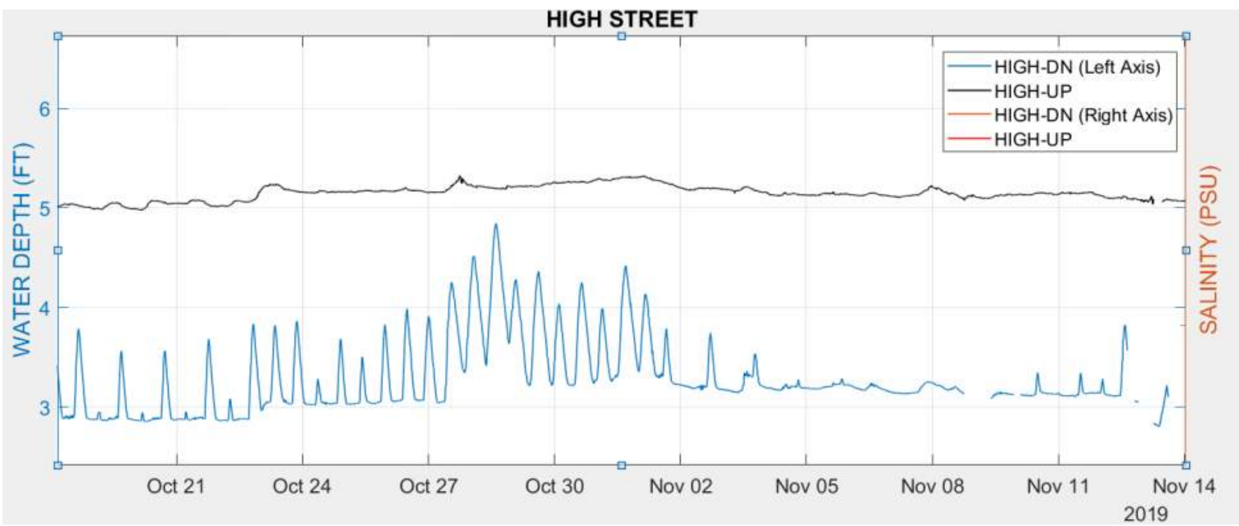


Figure 7: King tide event (approximately between October 28-November 2) as captured at these stations. The tidal downstream station saw a relatively small rise in water level

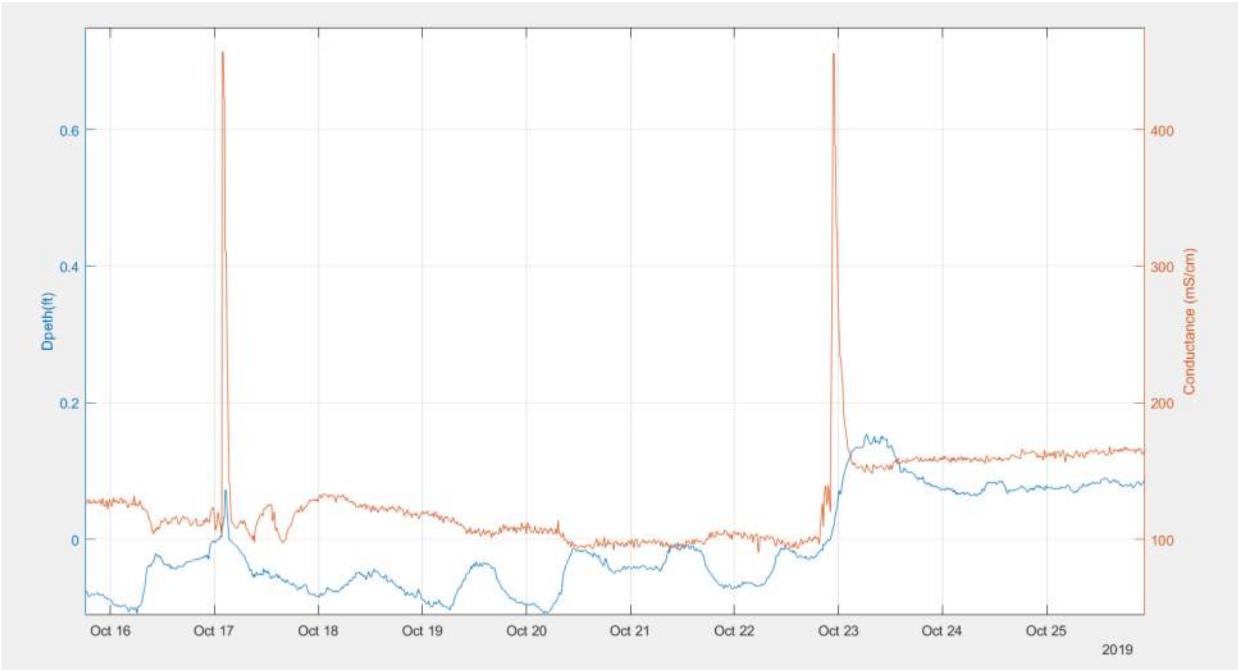


Figure 8:"Outliers" pointed out by Roy Schiff. These are not outliers, but what seems like real patterns in the data. So , they have not been filtered out.

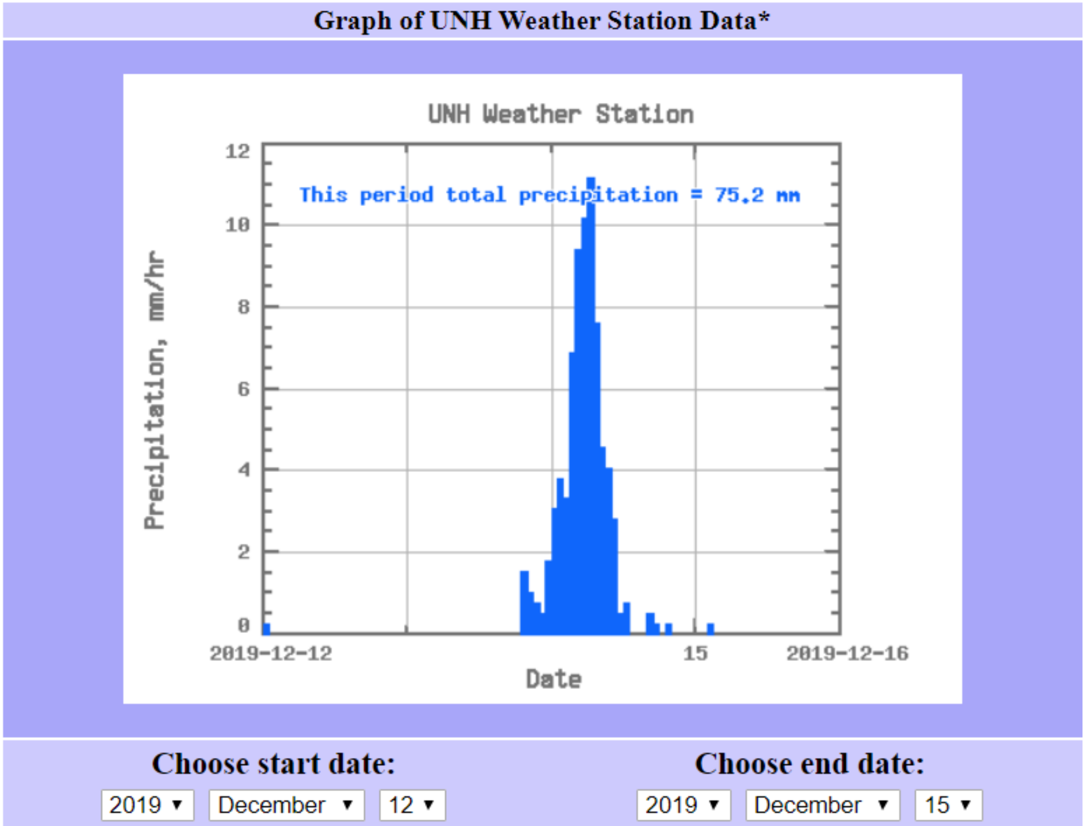


Figure 9: An extract of precipitation recorded during a mid-December storm at a Met station located on UNH campus. [Link]

References

Fofonoff, N. P., & Millard, R. C. (1983). Algorithms for computation of fundamental properties of seawater. *UNESCO Technical Papers in Marine Science*. https://doi.org/10.1111/j.1365-2486.2005.001000.x

Supplement Document for Meadow Pond Tide Data.

Gopal Mulukutla, April 2020

Accompanying Files:

1. Tide_Timing.CSV- file containing temporally aligned high and low tide times and depths of Meadow Pond tide monitoring stations with Hampton Harbor tide prediction
2. Tide_Constituents.xlx – Excel file containing out of harmonic analysis for 4 Meadow Pond stations and Hampton Habor tide prediction.

1.0 Harmonic Analysis

Harmonic analysis of tidal elevation data provides the underlying constituents that aggregate up to produce the data. Each constituent is indicative of a frequency that contributes to the overall signal, one of an endless number of periodic changes or variations in the relative positions of the Earth, Moon and Sun that influence tidal elevation. There are atleast 388 tidal constituents that have a named term associated with it, but far fewer than those are usually sufficient to predict tides upto useful accuracy. Table 1 below shows four principal diurnal and semidiurnal components. For a list of 37 principal tidal components that contribute to tide at Fort Point, NH, see the spreadsheet titled "Fort Point NH Tide" within the Tide_Constituents.XLX spreadsheet that accompanies this document.

1	M2	Principal lunar semidiurnal constituent
2	S2	Principal solar semidiurnal constituent
3	N2	Larger lunar elliptic semidiurnal constituent
4	K1	Lunar diurnal constituent

Hamornic analysis was performed on Meadown Pond tide elevation data and Hampton Harbor data to determine the principal constituents for each Tide constituent analysis and the variation of Form number. Two software different tools were used for this purpose.

1. UTIDE: a Unified Tidal Analysis and Prediction, by Daniel Codiga at the University of California, San Diego. http://www.po.gso.uri.edu/~codiga/utide/utide.htm
   This tool is an improvement n the T_tide (Pawlowicz et al, 2002) popularly used for harmonic analysis it is designed specifically for gappy or irregularly spaced data making it well suited for Meadow Pond Data
2. Tidalfit:A tool that used HAMELS (ordinary least squares) technique to fit tidal components to the detrended data. This tool was used to apply a fit onto the Hampton Harbor tide prediction. Since this tide prediction data is modeled output provided by NOAA, it provides only the time of high and or low tide for each day and is too sparse for direct use with UTIDE. An initial fit of the data was developed using Tidalfit, and the signal was resampled to hourly time intervals before anlyzing it using Tool #1 .
   https://www.mathworks.com/matlabcentral/fileexchange/19099-tidal-fitting-toolbox

2.0 Form number

Form number(F),is the ratio of the sums of diurnal(K1 andO1) and semidiurnal(M2 andS2) tide constituents. It is a common indicator to classify changes in periodic nature of tide (Thomas et al, 2019).

$$F = \frac{O1+K1}{M2+S2}$$

0 < F< 0.25 ➔ semidiurnal dominant tide

0.25< F 1.5 ➔ mixed semidiurnal tides

1.5< F< 3.0 ➔ mixed diurnal tides predominate

F>3.0 ➔diurnal tides predominate

Result of harmonic analysis are provided in the accompanying Excel file. Only the contributing amplitude for each constituent is provided in the table. Analysis was performed on stations with data that is reasonably complete and would provide a snapshot of tide at each road crossing. BROWN's river data was not selected for analysis due its incomplete nature. The chosen stations and their Form numbers are given below (Table 2).

Other variables that are needed to reconstruct the tidal elevation (such as phase and speed) are not provided in the analysis here. Form number analysis shows that Hampton Harbor is (predictably) predominantly semidiurnal tide but as it progresses upstream it transitions to a mixed semi diurnal tide. This is the result the increased bottom friction and constraints in flow.

Table 2: Variation in Form number starting at Hampton Harbor and at each road crossing

Station	**Form Number**	**M2/M4 ratio**
Hampton Harbor	0.16	0.007
RT101_DN	0.16	0.02
WIN_UP	0.42	0.34
HIGH_DN	0.49	0.49

3.0 M2/M4 ratio

In shallow estuaine type water bodies the principal lunar tide component (M2) is distorted by constraints in flow and bottom friction resulting in M4 overtides. The M2/M4 ratio profile along the marsh provides an understanding of the magnitude and extent of this distortion. Substantial increase in the M2/M4 ratio at WIN_UP suggests that most of the constraints in flow occurs upstream of RT101_DN and within the marsh system enclosed by the road crossings. Further understanding of the M4 distortion and the nonlinear response of the estuary is ongoing. Among the factors that need to be looked at are the asymmetric distortion of the tide curve due to the manifestation of M4 overtides. This causes differences in duration of the ebb and flow signals and their variation with predicted Hampton Harbor tide will provide an understanding of tidal hydrodynamics within the marsh system.

4.0 Lag in Tidal Peaks

Distortion of the tidal curve as it passes through the tidal marsh system results in a lagged high and low tide response in comparison with Hampton Harbor tide. This lag in response was quantified at each road crossing and Brown's river. This data is summarized in plots in the next few pages. Underlying data is provided as an accompanying file - Tide_Timing. CSV file. The process of extracting this data is briefly described below.

Step 1: Select data from stations Hampton Harbor, HIGH-DN, WIN-UP, RT101-DN and BROWNS

Step 2: For the selected Meadow Pond stations determine the peak and trough for each day, identify either as a high tide or low tide event and note the time of each event. Remove outliers or peaks or troughs identified in days with incomplete data.

Step 3: Cycle through each detected peak and trough for a station and identify the tidal cycle it belongs in relation with Hampton Habor, and quantify the difference in time between corresponding high or low tide time reported at the harbor. Plot data and develop regressions for each station.

Step 4: Compile data of times of peaks and trough with Hampton Harbor tide table.

Please check the following plots for results. Each page contains 2 plots, showing the Hampton Harbor tide depth (x-axis) plotted against the time difference in tide between the said station and the harbor for that tide cycle. There are four stations for which the plots were made (in order HIGH_UP, WIN_UP, RT101_DN and BROWN). Parameters for best linear fit are also provided.

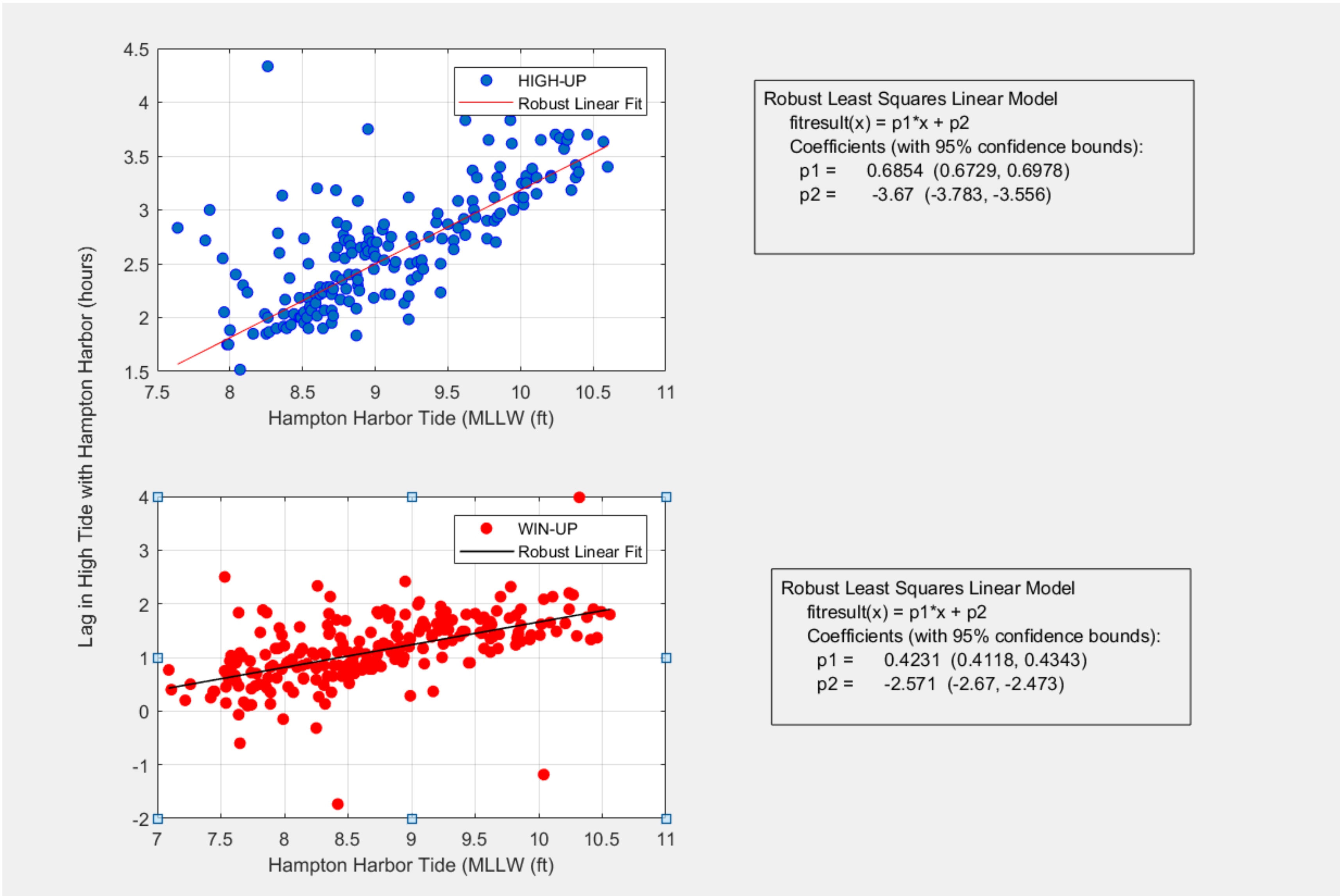
Caveat : data for BROWN is sparse, and as such data show weak relationships.

References

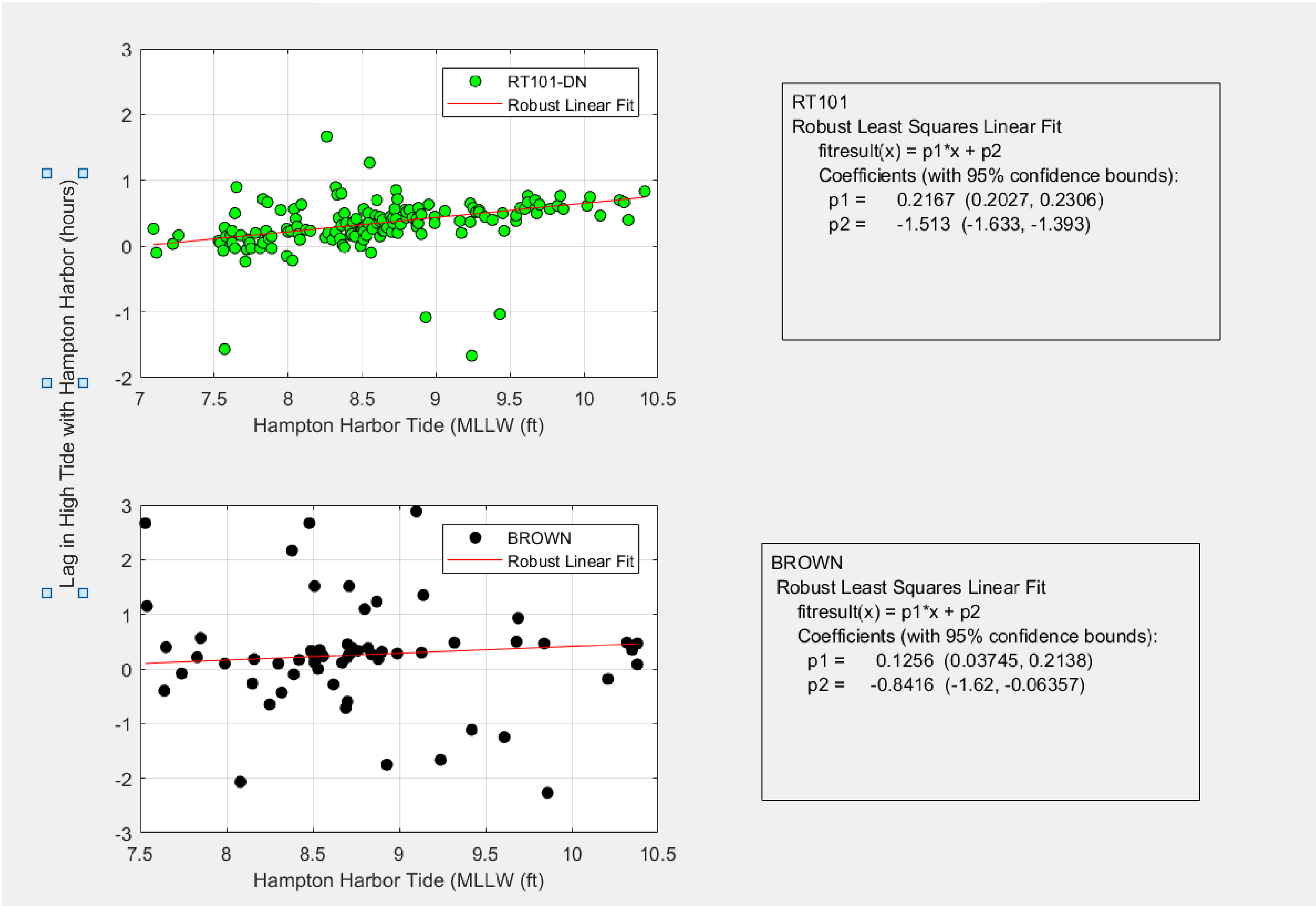
R. Pawlowicz, B. Beardsley, and S. Lentz, Classical tidal harmonic analysis including error estimates in MATLAB using T_TIDE, Computers and Geosciences 28 (2002), 929-937.

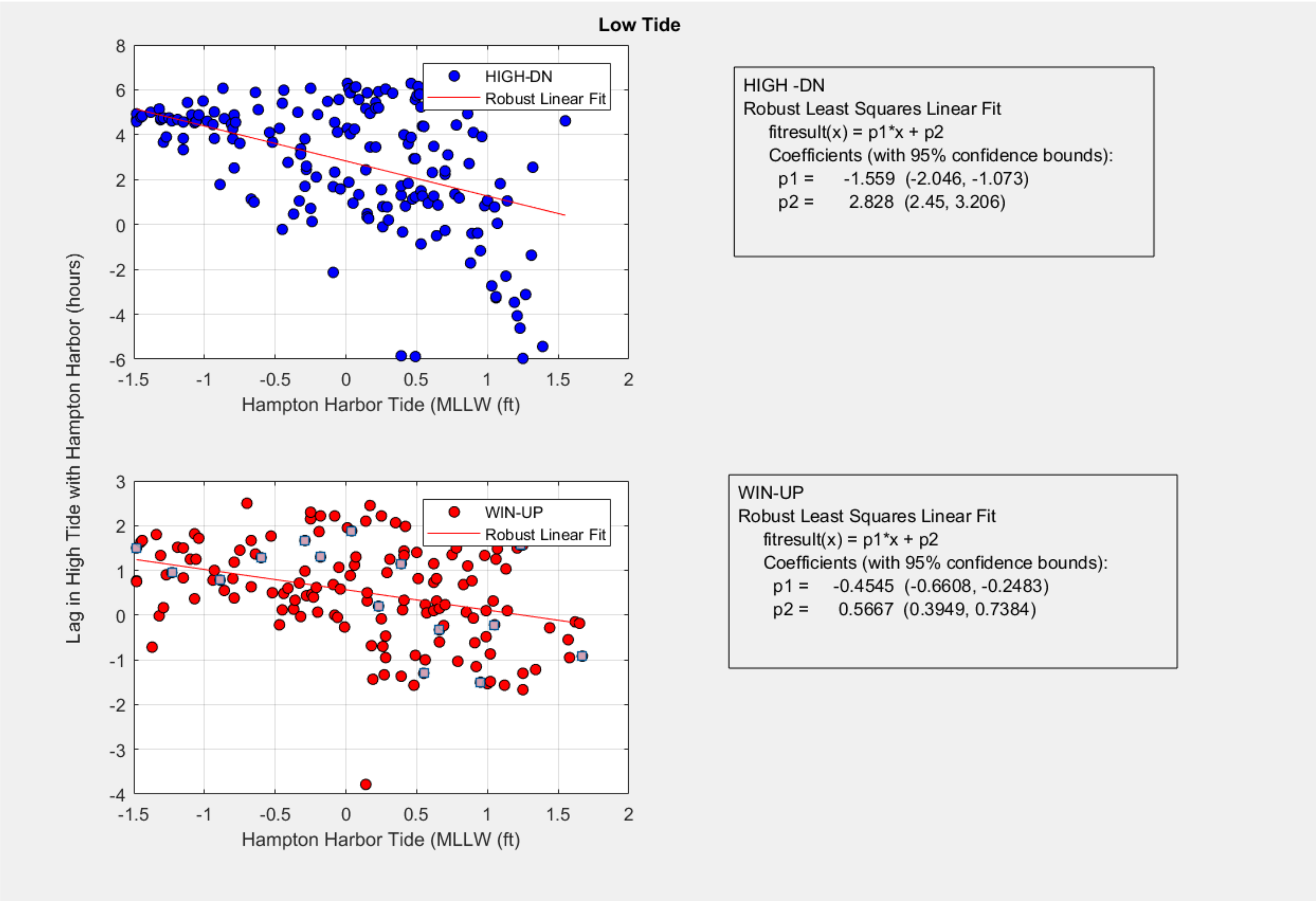
Thomas,J.;Velamala,S.N.,andPrasad,K.V.S.R.,2019.Numerical simulation of tidal constituents in Thane Creek and the Ulhas estuary,west coast of India. Journal of Coastal Research, 35(2), 376–388.

High Tide

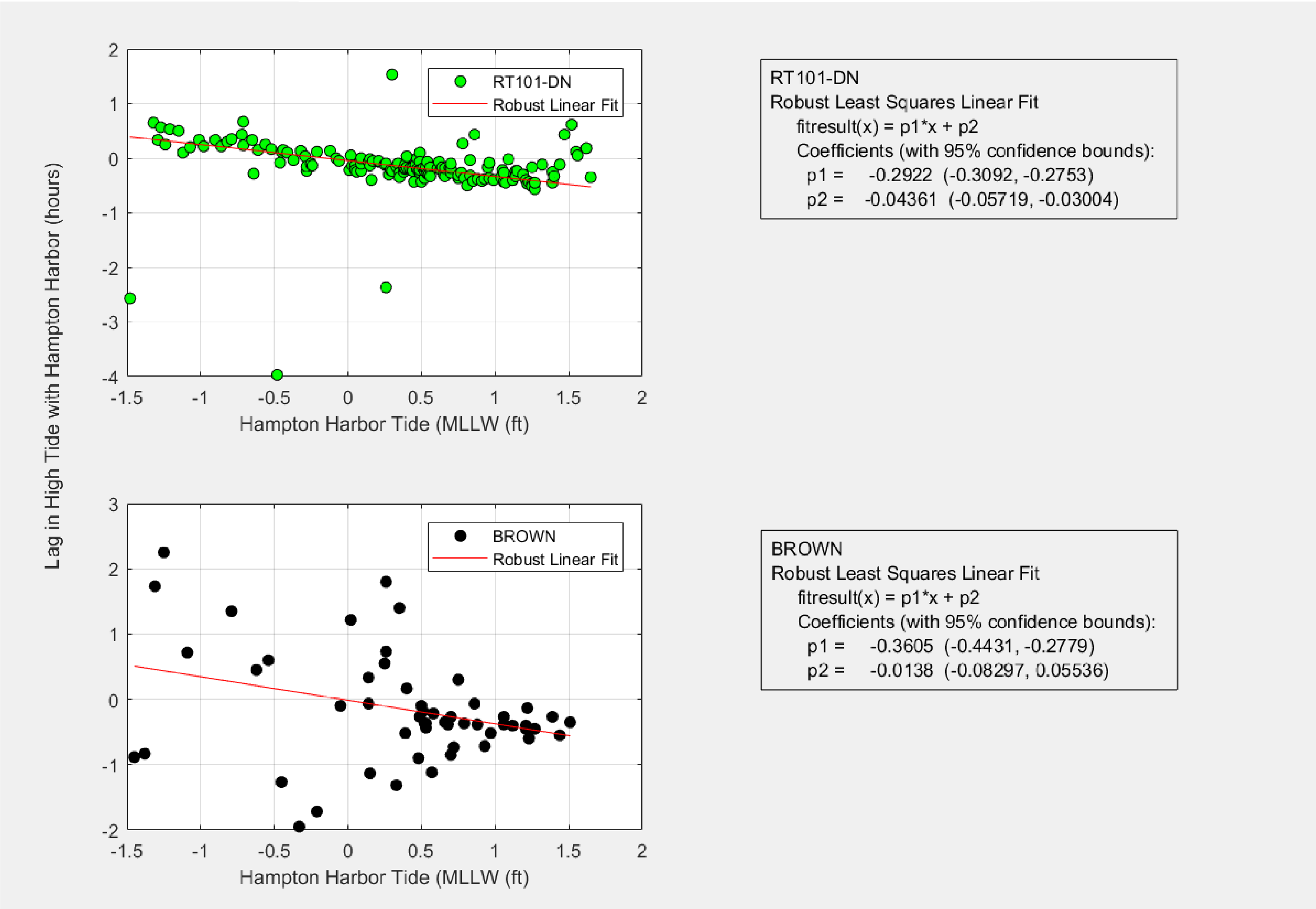

HIGH-UP
Robust Linear Fit
WIN-UP
Robust Linear Fit
Lag in High Tide with Hampton Harbor (hours)
Hampton Harbor Tide (MLLW (ft)
Hampton Harbor Tide (MLLW (ft)
Robust Least Squares Linear Model
fitresult(x) = p1*x + p2
Coefficients (with 95% confidence bounds):
p1 = 0.6854 (0.6729, 0.6978)
p2 = -3.67 (-3.783, -3.556)
Robust Least Squares Linear Model
fitresult(x) = p1*x + p2
Coefficients (with 95% confidence bounds):
p1 = 0.4231 (0.4118, 0.4343)
p2 = -2.571 (-2.67, -2.473)

High Tide

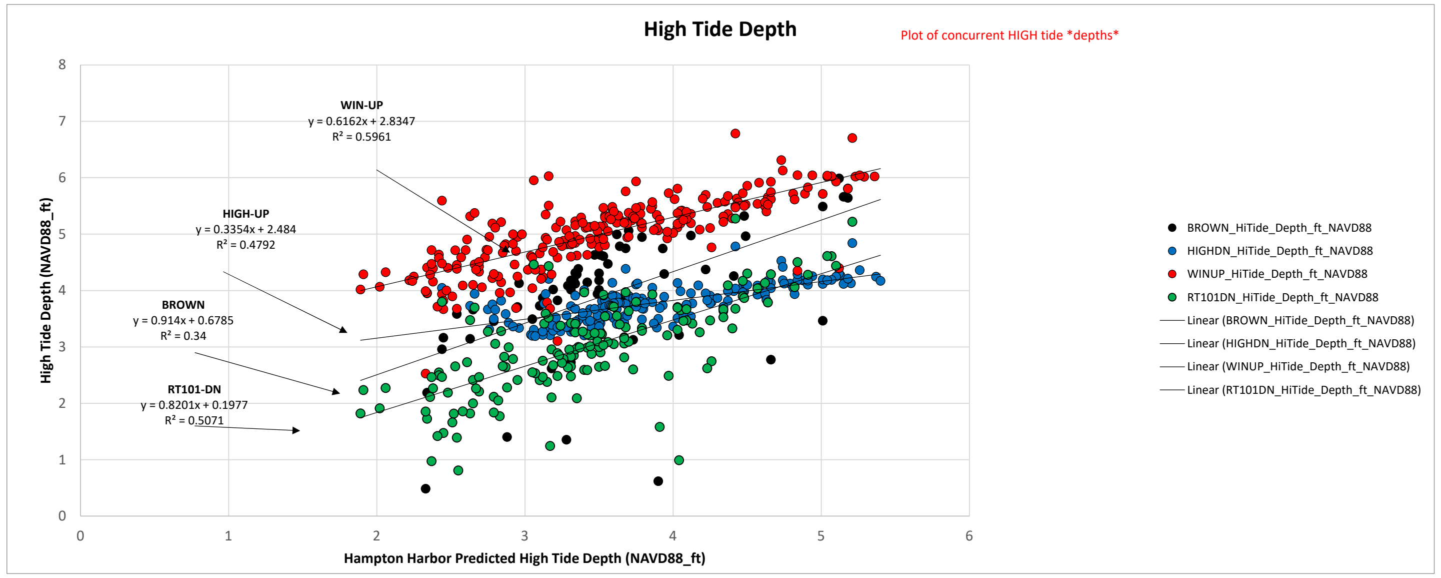

RT101-DN
Robust Linear Fit
Lag in High Tide with Hampton Harbor (hours)
Hampton Harbor Tide (MLLW (ft)
RT101
Robust Least Squares Linear Fit
fitresult(x) = p1*x + p2
Coefficients (with 95% confidence bounds):
p1 = 0.2167 (0.2027, 0.2306)
p2 = -1.513 (-1.633, -1.393)
BROWN
Robust Linear Fit
Hampton Harbor Tide (MLLW (ft)
BROWN
Robust Least Squares Linear Fit
fitresult(x) = p1*x + p2
Coefficients (with 95% confidence bounds):
p1 = 0.1256 (0.03745, 0.2138)
p2 = -0.8416 (-1.62, -0.06357)

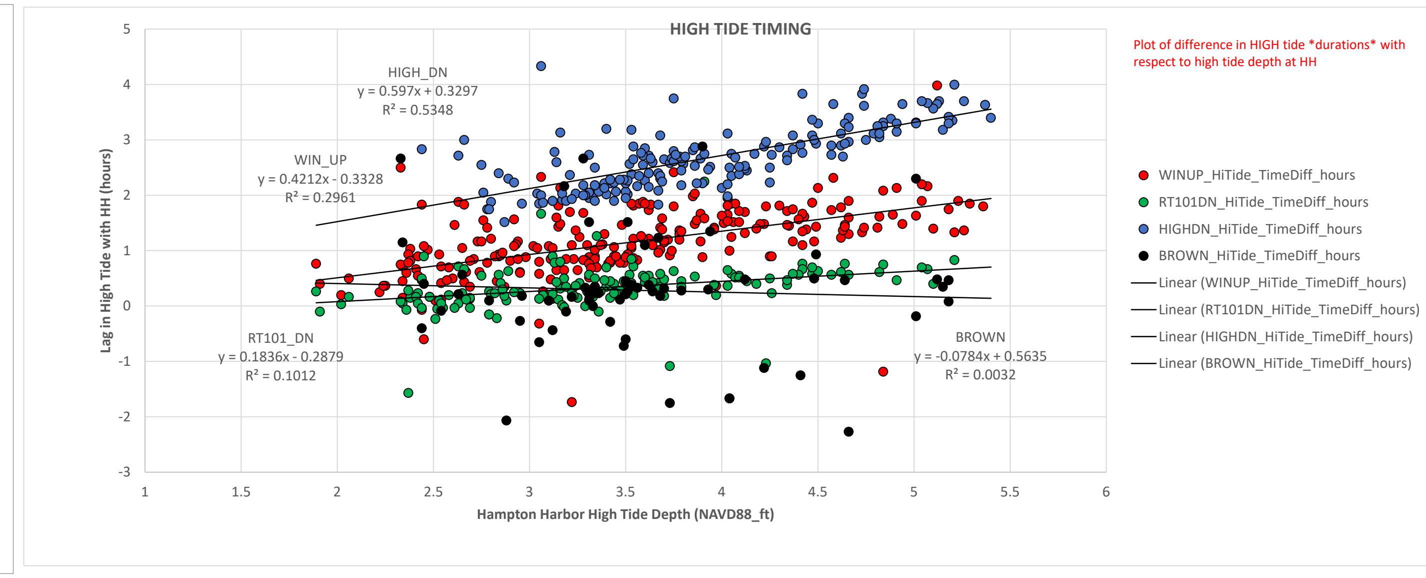

Low Tide
HIGH-DN
Robust Linear Fit
Lag in High Tide with Hampton Harbor (hours)
Hampton Harbor Tide (MLLW (ft)
8
6
4
2
0
-2
-4
-6
-1.5
-1
-0.5
0
0.5
1
1.5
2
HIGH -DN
Robust Least Squares Linear Fit
fitresult(x) = p1*x + p2
Coefficients (with 95% confidence bounds):
p1 = -1.559 (-2.046, -1.073)
p2 = 2.828 (2.45, 3.206)
WIN-UP
Robust Linear Fit
3
2
1
0
-1
-2
-3
-4
Hampton Harbor Tide (MLLW (ft)
WIN-UP
Robust Least Squares Linear Fit
fitresult(x) = p1*x + p2
Coefficients (with 95% confidence bounds):
p1 = -0.4545 (-0.6608, -0.2483)
p2 = 0.5667 (0.3949, 0.7384)

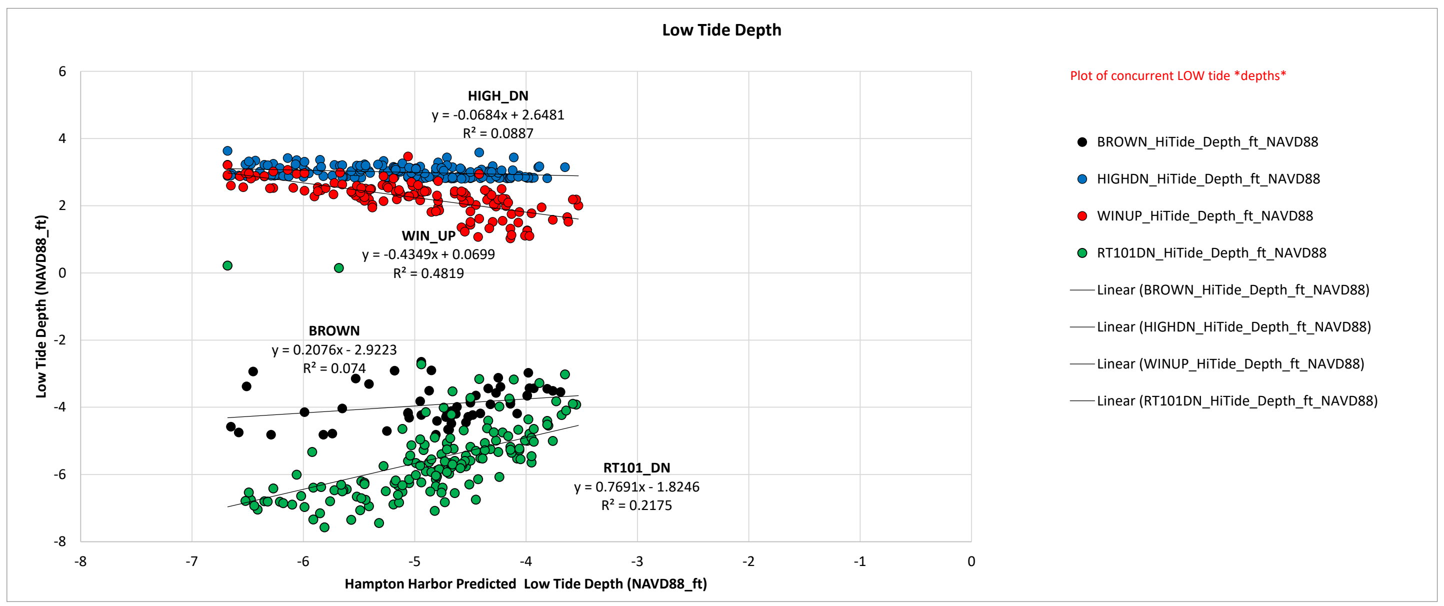
Low Tide

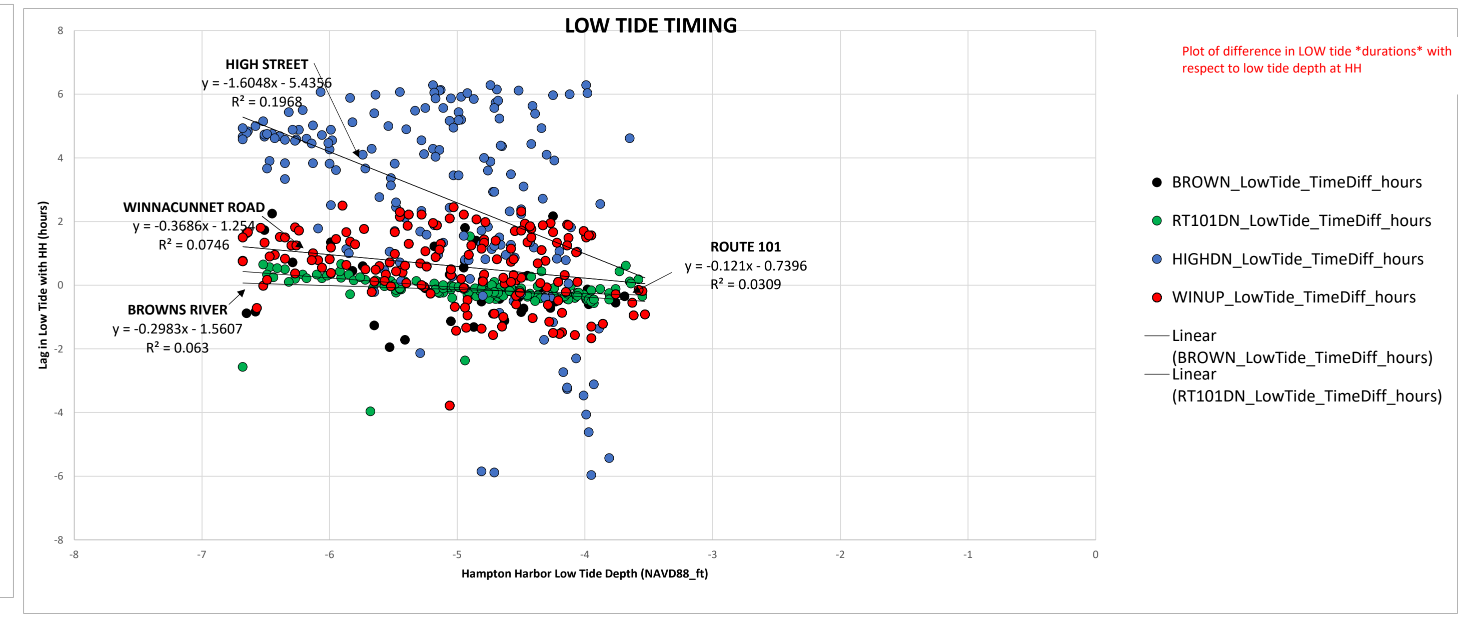


Lag in High Tide with Hampton Harbor (hours)
Hampton Harbor Tide (MLLW (ft)
RT101-DN
Robust Linear Fit
RT101-DN
Robust Least Squares Linear Fit
fitresult(x) = p1*x + p2
Coefficients (with 95% confidence bounds):
p1 = -0.2922 (-0.3092, -0.2753)
p2 = -0.04361 (-0.05719, -0.03004)
Hampton Harbor Tide (MLLW (ft)
BROWN
Robust Linear Fit
BROWN
Robust Least Squares Linear Fit
fitresult(x) = p1*x + p2
Coefficients (with 95% confidence bounds):
p1 = -0.3605 (-0.4431, -0.2779)
p2 = -0.0138 (-0.08297, 0.05536)


High Tide Depth
Plot of concurrent HIGH tide *depths*
WIN-UP
y = 0.6162x + 2.8347
R² = 0.5961
HIGH-UP
y = 0.3354x + 2.484
R² = 0.4792
BROWN
y = 0.914x + 0.6785
R² = 0.34
RT101-DN
y = 0.8201x + 0.1977
R² = 0.5071
High Tide Depth (NAVD88_ft)
Hampton Harbor Predicted High Tide Depth (NAVD88_ft)
BROWN_HiTide_Depth_ft_NAVD88
HIGHDN_HiTide_Depth_ft_NAVD88
WINUP_HiTide_Depth_ft_NAVD88
RT101DN_HiTide_Depth_ft_NAVD88
Linear (BROWN_HiTide_Depth_ft_NAVD88)
Linear (HIGHDN_HiTide_Depth_ft_NAVD88)
Linear (WINUP_HiTide_Depth_ft_NAVD88)
Linear (RT101DN_HiTide_Depth_ft_NAVD88)


HIGH TIDE TIMING
Plot of difference in HIGH tide *durations* with respect to high tide depth at HH
HIGH_DN
y = 0.597x + 0.3297
R² = 0.5348
WIN_UP
y = 0.4212x - 0.3328
R² = 0.2961
RT101_DN
y = 0.1836x - 0.2879
R² = 0.1012
BROWN
y = -0.0784x + 0.5635
R² = 0.0032
Lag in High Tide with HH (hours)
Hampton Harbor High Tide Depth (NAVD88_ft)
WINUP_HiTide_TimeDiff_hours
RT101DN_HiTide_TimeDiff_hours
HIGHDN_HiTide_TimeDiff_hours
BROWN_HiTide_TimeDiff_hours
Linear (WINUP_HiTide_TimeDiff_hours)
Linear (RT101DN_HiTide_TimeDiff_hours)
Linear (HIGHDN_HiTide_TimeDiff_hours)
Linear (BROWN_HiTide_TimeDiff_hours)


Low Tide Depth
Plot of concurrent LOW tide *depths*
HIGH_DN
y = -0.0684x + 2.6481
R² = 0.0887
WIN_UP
y = -0.4349x + 0.0699
R² = 0.4819
BROWN
y = 0.2076x - 2.9223
R² = 0.074
RT101_DN
y = 0.7691x - 1.8246
R² = 0.2175
Low Tide Depth (NAVD88_ft)
Hampton Harbor Predicted Low Tide Depth (NAVD88_ft)
BROWN_HiTide_Depth_ft_NAVD88
HIGHDN_HiTide_Depth_ft_NAVD88
WINUP_HiTide_Depth_ft_NAVD88
RT101DN_HiTide_Depth_ft_NAVD88
Linear (BROWN_HiTide_Depth_ft_NAVD88)
Linear (HIGHDN_HiTide_Depth_ft_NAVD88)
Linear (WINUP_HiTide_Depth_ft_NAVD88)
Linear (RT101DN_HiTide_Depth_ft_NAVD88)

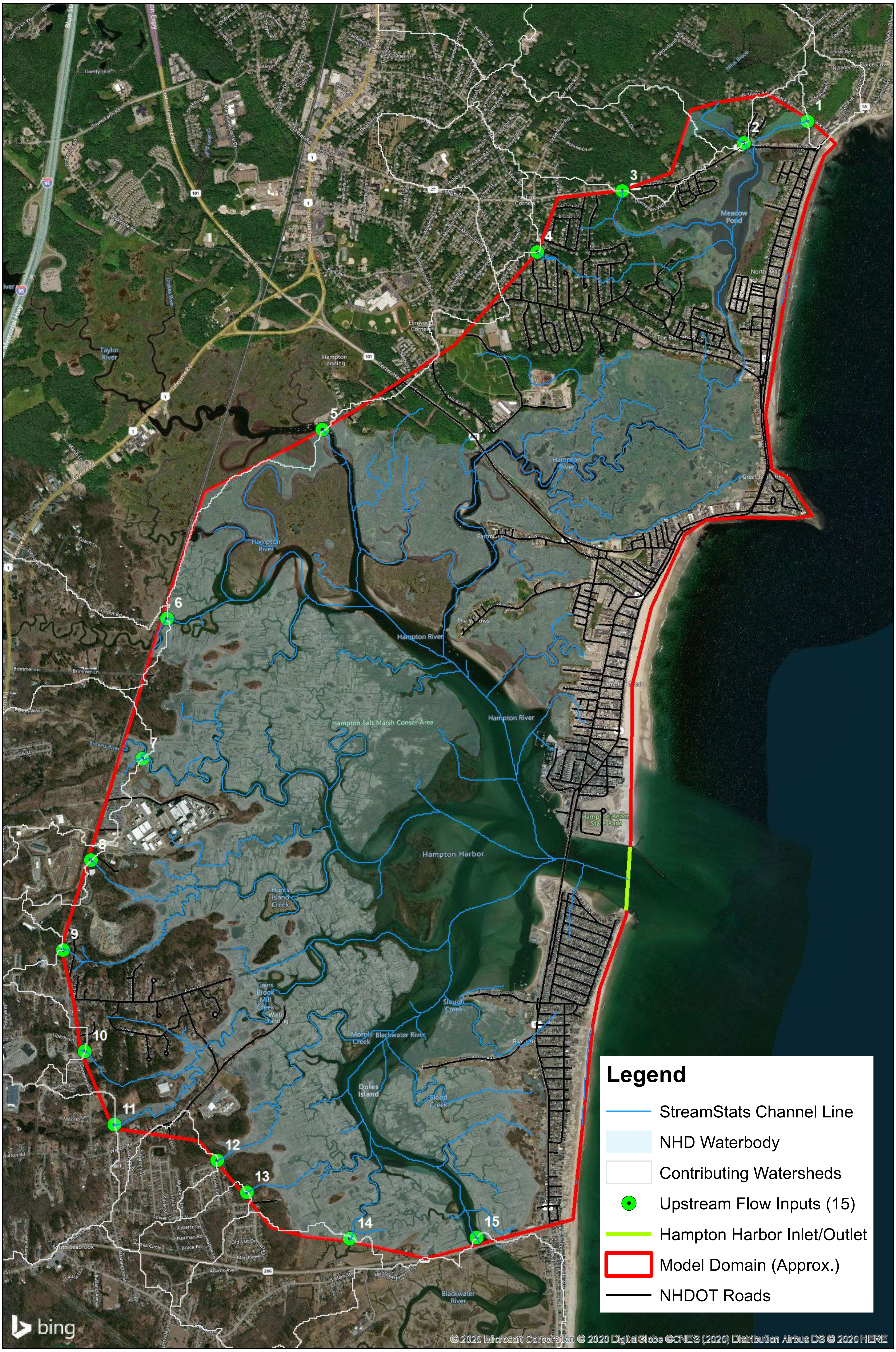

LOW TIDE TIMING
Plot of difference in LOW tide *durations* with respect to low tide depth at HH
HIGH STREET
y = -1.6048x - 5.4356
R² = 0.1968
WINNACUNNET ROAD
y = -0.3686x - 1.254
R² = 0.0746
BROWNS RIVER
y = -0.2983x - 1.5607
R² = 0.063
ROUTE 101
y = -0.121x - 0.7396
R² = 0.0309
Lag in Low Tide with HH (hours)
Hampton Harbor Low Tide Depth (NAVD88_ft)
BROWN_LowTide_TimeDiff_hours
RT101DN_LowTide_TimeDiff_hours
HIGHDN_LowTide_TimeDiff_hours
WINUP_LowTide_TimeDiff_hours
Linear (BROWN_LowTide_TimeDiff_hours)
Linear (RT101DN_LowTide_TimeDiff_hours)

APPENDIX D

ESTIMATED FRESHWATER AND TIDAL HYDROLOGY INPUTS

Engineering Report

March 2021



HYDRAULIC MODEL SETUP
HAMPTON FLOOD STUDY
TOWN OF HAMPTON, NH
(MMI, 2021)

0 400 800 1,600 2,400 3,200
Feet
1 in = 2,000 feet

N

MILONE & MACBROOM
1 South Main St
Waterbury, VT 05676
802-882-8335

(MMI, 2021)

Peak Flow Estimates from StreamStats Batch Processor (See GIS points and geodatabase)
Hampton Flood Study
4/7/2020

Peak Flow Estimates

Flow Input ID	Flow Input Name	Drainage Area (Square Miles)	2 Year Peak Flood (cfs)	5 Year Peak Flood	10 Year Peak Flood	25 Year Peak Flood	50 Year Peak Flood	100 Year Peak Flood	500 Year Peak Flood
1	Unnamed Tributary	0.1	3	6	8	12	16	20	32
2	Nilus Brook	0.8	6	10	14	20	24	30	45
3	Unnamed Tributary	0.4	11	20	29	40	51	63	96
4	Developed Area	0.2	13	24	34	49	62	79	124
5	Hampton River	15.9	196	318	419	555	668	806	1,150
6	Hampton Falls River	6.9	106	177	237	317	384	466	666
7	Browns River	0.7	17	30	41	58	72	90	136
8	Unnamed Tributary	0.1	4	8	12	18	22	29	45
9	Hunts Island Creek	0.3	14	26	38	54	68	86	135
10	Unnamed Tributary	0.2	7	14	19	28	36	46	72
11	Cains Brook Mill Creek	2.7	84	144	197	271	334	412	611
12	Morrills Brook	0.2	7	13	18	26	34	43	67
13	Unnamed Tributary	0.1	3	6	8	12	15	19	30
14	Unnamed Tributary	0.3	2	3	4	6	8	10	15
15	Blackwater River	7.4	21	34	45	59	71	86	121

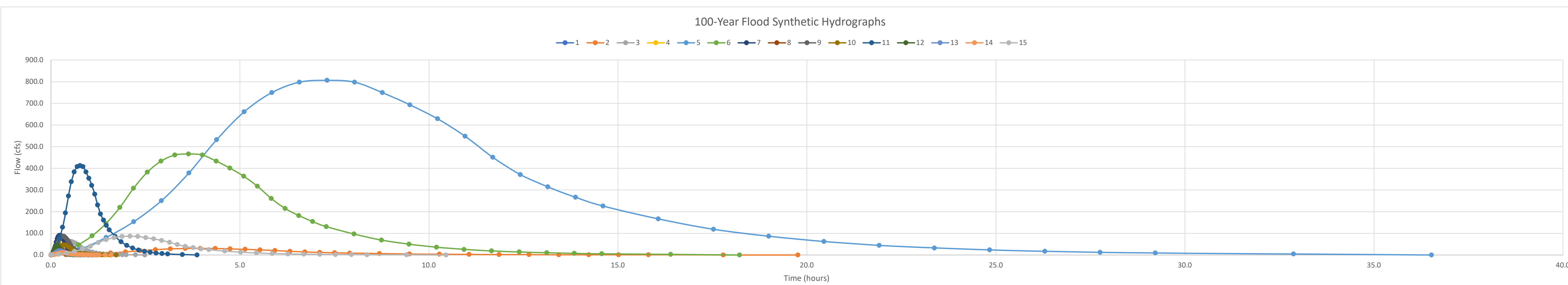
(MMI, 2021)

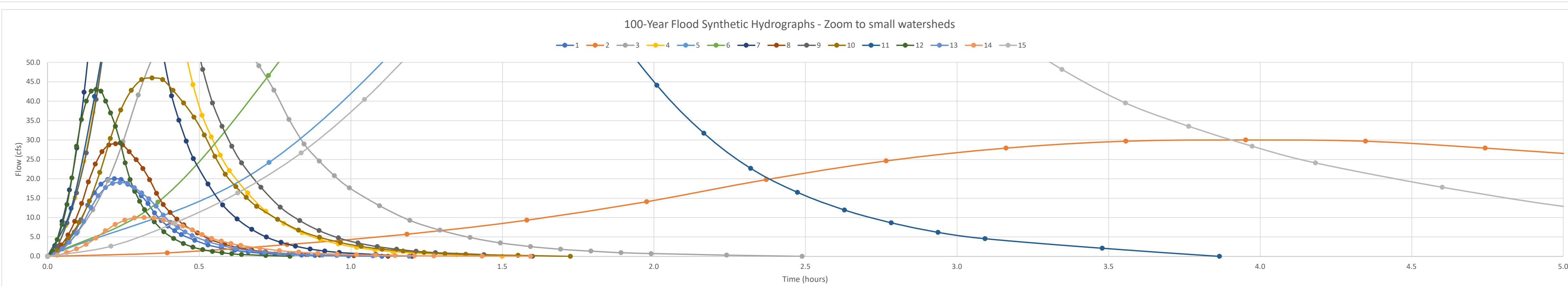
Estimated 100-Year Flood Hydrographs
NRCS Synthetic Hydrographs with USGS StreamStats Peaks and Tc Calculations
5/15/2020

Flow Input ID	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Flow Input Name	Unnamed Tributary	Nilus Brook	Unnamed Tributary	Developed Area	Hampton River	Hampton Falls River	Browns River	Unnamed Tributary	Hunts Island Creek	Unnamed Tributary	Cains Brook Mill Creek	Morrills Brook	Unnamed Tributary	Unnamed Tributary	Blackwater River
Drainage Area (Square Miles)	0.1	0.8	0.4	0.2	15.9	6.9	0.7	0.1	0.3	0.2	2.7	0.2	0.1	0.3	7.4
100-Year Flood Peak (cfs)	20	30	63	79	806	466	90	29	86	46	412	43	19	10	86
Tc (hours)	0.3	5.9	0.7	0.5	11.0	5.5	0.4	0.3	0.5	0.5	1.2	0.2	0.4	0.5	3.1
Lag (hours)	0.2	3.6	0.4	0.3	6.6	3.3	0.2	0.2	0.3	0.3	0.7	0.1	0.2	0.3	1.9
Duration (hours)	0.04	0.79	0.10	0.06	1.46	0.73	0.05	0.04	0.06	0.07	0.15	0.03	0.05	0.06	0.42
Tp (hours)	0.2	4.0	0.5	0.3	7.3	3.6	0.2	0.2	0.3	0.3	0.8	0.2	0.2	0.3	2.1

NRCS Unit Hydrograph		1		2		3		4		5		6		7	
T/Tp	Q/Qp	T (hours)	Q (cfs)	T (hours)	Q (cfs)	T (hours)	Q (cfs)	T (hours)	Q (cfs)	T (hours)	Q (cfs)	T (hours)	Q (cfs)	T (hours)	Q (cfs)
0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.1	0.03	0.0	0.6	0.4	0.9	0.0	1.9	0.0	2.4	0.7	24.2	0.4	14.0	0.0	2.7
0.2	0.1	0.0	2.0	0.8	3.0	0.1	6.3	0.1	7.9	1.5	80.6	0.7	46.6	0.0	9.0
0.3	0.19	0.1	3.8	1.2	5.7	0.1	12.0	0.1	15.0	2.2	153.1	1.1	88.5	0.1	17.1
0.4	0.31	0.1	6.2	1.6	9.3	0.2	19.5	0.1	24.5	2.9	249.9	1.5	144.5	0.1	27.9
0.5	0.47	0.1	9.4	2.0	14.1	0.2	29.6	0.1	37.1	3.7	378.8	1.8	219.0	0.1	42.3
0.6	0.66	0.1	13.2	2.4	19.8	0.3	41.6	0.2	52.1	4.4	532.0	2.2	307.6	0.1	59.4
0.7	0.82	0.2	16.4	2.8	24.6	0.3	51.7	0.2	64.8	5.1	660.9	2.6	382.1	0.2	73.8
0.8	0.93	0.2	18.6	3.2	27.9	0.4	58.6	0.2	73.5	5.8	749.6	2.9	433.4	0.2	83.7
0.9	0.99	0.2	19.8	3.6	29.7	0.4	62.4	0.3	78.2	6.6	797.9	3.3	461.3	0.2	89.1
1	1	0.2	20.0	4.0	30.0	0.5	63.0	0.3	79.0	7.3	806.0	3.6	466.0	0.2	90.0
1.1	0.99	0.2	19.8	4.3	29.7	0.5	62.4	0.3	78.2	8.0	797.9	4.0	461.3	0.3	89.1
1.2	0.93	0.3	18.6	4.7	27.9	0.6	58.6	0.4	73.5	8.8	749.6	4.4	433.4	0.3	83.7
1.3	0.86	0.3	17.2	5.1	25.8	0.6	54.2	0.4	67.9	9.5	693.2	4.7	400.8	0.3	77.4
1.4	0.78	0.3	15.6	5.5	23.4	0.7	49.1	0.4	61.6	10.2	628.7	5.1	363.5	0.3	70.2
1.5	0.68	0.3	13.6	5.9	20.4	0.7	42.8	0.4	53.7	11.0	548.1	5.5	316.9	0.4	61.2
1.6	0.56	0.4	11.2	6.3	16.8	0.8	35.3	0.5	44.2	11.7	451.4	5.8	261.0	0.4	50.4
1.7	0.46	0.4	9.2	6.7	13.8	0.8	29.0	0.5	36.3	12.4	370.8	6.2	214.4	0.4	41.4
1.8	0.39	0.4	7.8	7.1	11.7	0.9	24.6	0.5	30.8	13.1	314.3	6.6	181.7	0.4	35.1
1.9	0.33	0.4	6.6	7.5	9.9	0.9	20.8	0.6	26.1	13.9	266.0	6.9	153.8	0.5	29.7
2	0.28	0.4	5.6	7.9	8.4	1.0	17.6	0.6	22.1	14.6	225.7	7.3	130.5	0.5	25.2
2.2	0.207	0.5	4.1	8.7	6.2	1.1	13.0	0.7	16.4	16.1	166.8	8.0	96.5	0.5	18.6
2.4	0.147	0.5	2.9	9.5	4.4	1.2	9.3	0.7	11.6	17.5	118.5	8.7	68.5	0.6	13.2
2.6	0.107	0.6	2.1	10.3	3.2	1.3	6.7	0.8	8.5	19.0	86.2	9.5	49.9	0.6	9.6
2.8	0.077	0.6	1.5	11.1	2.3	1.4	4.9	0.8	6.1	20.5	62.1	10.2	35.9	0.7	6.9
3	0.055	0.7	1.1	11.9	1.7	1.5	3.5	0.9	4.3	21.9	44.3	10.9	25.6	0.7	5.0
3.2	0.04	0.7	0.8	12.6	1.2	1.6	2.5	1.0	3.2	23.4	32.2	11.7	18.6	0.8	3.6
3.4	0.029	0.8	0.6	13.4	0.9	1.7	1.8	1.0	2.3	24.8	23.4	12.4	13.5	0.8	2.6
3.6	0.021	0.8	0.4	14.2	0.6	1.8	1.3	1.1	1.7	26.3	16.9	13.1	9.8	0.9	1.9
3.8	0.015	0.8	0.3	15.0	0.5	1.9	0.9	1.1	1.2	27.8	12.1	13.8	7.0	0.9	1.4
4	0.011	0.9	0.2	15.8	0.3	2.0	0.7	1.2	0.9	29.2	8.9	14.6	5.1	1.0	1.0
4.5	0.005	1.0	0.1	17.8	0.2	2.2	0.3	1.3	0.4	32.9	4.0	16.4	2.3	1.1	0.5
5	0	1.1	0.0	19.8	0.0	2.5	0.0	1.5	0.0	36.5	0.0	18.2	0.0	1.2	0.0

NRCS Unit Hydrograph		8		9		10		11		12		13		14		15	
T/Tp	Q/Qp	T (hours)	Q (cfs)	T (hours)	Q (cfs)	T (hours)	Q (cfs)	T (hours)	Q (cfs)	T (hours)	Q (cfs)	T (hours)	Q (cfs)	T (hours)	Q (cfs)	T (hours)	Q (cfs)
0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.1	0.03	0.0	0.9	0.0	2.6	0.0	1.4	0.1	12.4	0.0	1.3	0.0	0.6	0.0	0.3	0.2	2.6
0.2	0.1	0.0	2.9	0.1	8.6	0.1	4.6	0.2	41.2	0.0	4.3	0.0	1.9	0.1	1.0	0.4	8.6
0.3	0.19	0.1	5.5	0.1	16.3	0.1	8.7	0.2	78.3	0.0	8.2	0.1	3.6	0.1	1.9	0.6	16.3
0.4	0.31	0.1	9.0	0.1	26.7	0.1	14.3	0.3	127.7	0.1	13.3	0.1	5.9	0.1	3.1	0.8	26.7
0.5	0.47	0.1	13.6	0.2	40.4	0.2	21.6	0.4	193.6	0.1	20.2	0.1	8.9	0.2	4.7	1.0	40.4
0.6	0.66	0.1	19.1	0.2	56.8	0.2	30.4	0.5	271.9	0.1	28.4	0.1	12.5	0.2	6.6	1.3	56.8
0.7	0.82	0.2	23.8	0.2	70.5	0.2	37.7	0.5	337.8	0.1	35.3	0.2	15.6	0.2	8.2	1.5	70.5
0.8	0.93	0.2	27.0	0.3	80.0	0.3	42.8	0.6	383.2	0.1	40.0	0.2	17.7	0.3	9.3	1.7	80.0
0.9	0.99	0.2	28.7	0.3	85.1	0.3	45.5	0.7	407.9	0.1	42.6	0.2	18.8	0.3	9.9	1.9	85.1
1	1	0.2	29.0	0.3	86.0	0.3	46.0	0.8	412.0	0.2	43.0	0.2	19.0	0.3	10.0	2.1	86.0
1.1	0.99	0.2	28.7	0.4	85.1	0.4	45.5	0.9	407.9	0.2	42.6	0.3	18.8	0.4	9.9	2.3	85.1
1.2	0.93	0.3	27.0	0.4	80.0	0.4	42.8	0.9	383.2	0.2	40.0	0.3	17.7	0.4	9.3	2.5	80.0
1.3	0.86	0.3	24.9	0.4	74.0	0.4	39.6	1.0	354.3	0.2	37.0	0.3	16.3	0.4	8.6	2.7	74.0
1.4	0.78	0.3	22.6	0.4	67.1	0.5	35.9	1.1	321.4	0.2	33.5	0.3	14.8	0.4	7.8	2.9	67.1
1.5	0.68	0.3	19.7	0.5	58.5	0.5	31.3	1.2	280.2	0.2	29.2	0.4	12.9	0.5	6.8	3.1	58.5
1.6	0.56	0.4	16.2	0.5	48.2	0.6	25.8	1.2	230.7	0.3	24.1	0.4	10.6	0.5	5.6	3.3	48.2
1.7	0.46	0.4	13.3	0.5	39.6	0.6	21.2	1.3	189.5	0.3	19.8	0.4	8.7	0.5	4.6	3.6	39.6
1.8	0.39	0.4	11.3	0.6	33.5	0.6	17.9	1.4	160.7	0.3	16.8	0.4	7.4	0.6	3.9	3.8	33.5
1.9	0.33	0.4	9.6	0.6	28.4	0.7	15.2	1.5	136.0	0.3	14.2	0.5	6.3	0.6	3.3	4.0	28.4
2	0.28	0.4	8.1	0.6	24.1	0.7	12.9	1.5	115.4	0.3	12.0	0.5	5.3	0.6	2.8	4.2	24.1
2.2	0.207	0.5	6.0	0.7	17.8	0.8	9.5	1.7	85.3	0.4	8.9	0.5	3.9	0.7	2.1	4.6	17.8
2.4	0.147	0.5	4.3	0.8	12.6	0.8	6.8	1.9	60.6	0.4	6.3	0.6	2.8	0.8	1.5	5.0	12.6
2.6	0.107	0.6	3.1	0.8	9.2	0.9	4.9	2.0	44.1	0.4	4.6	0.6	2.0	0.8	1.1	5.4	9.2
2.8	0.077	0.6	2.2	0.9	6.6	1.0	3.5	2.2	31.7	0.4	3.3	0.7	1.5	0.9	0.8	5.9	6.6
3	0.055	0.7	1.6	1.0	4.7	1.0	2.5	2.3	22.7	0.5	2.4	0.7	1.0	1.0	0.6	6.3	4.7
3.2	0.04	0.7	1.2	1.0	3.4	1.1	1.8	2.5	16.5	0.5	1.7	0.8	0.8	1.0	0.4	6.7	3.4
3.4	0.029	0.8	0.8	1.1	2.5	1.2	1.3	2.6	11.9	0.5	1.2	0.8	0.6	1.1	0.3	7.1	2.5
3.6	0.021	0.8	0.6	1.2	1.8	1.2	1.0	2.8	8.7	0.6	0.9	0.9	0.4	1.1	0.2	7.5	1.8
3.8	0.015	0.9	0.4	1.2	1.3	1.3	0.7	2.9	6.2	0.6	0.6	0.9	0.3	1.2	0.2	7.9	1.3
4	0.011	0.9	0.3	1.3	0.9	1.4	0.5	3.1	4.5	0.6	0.5	1.0	0.2	1.3	0.1	8.4	0.9
4.5	0.005	1.0	0.1	1.4	0.4	1.6	0.2	3.5	2.1	0.7	0.2	1.1	0.1	1.4	0.1	9.4	0.4
5	0	1.1	0.0	1.6	0.0	1.7	0.0	3.9	0.0	0.8	0.0	1.2	0.0	1.6	0.0	10.5	0.0





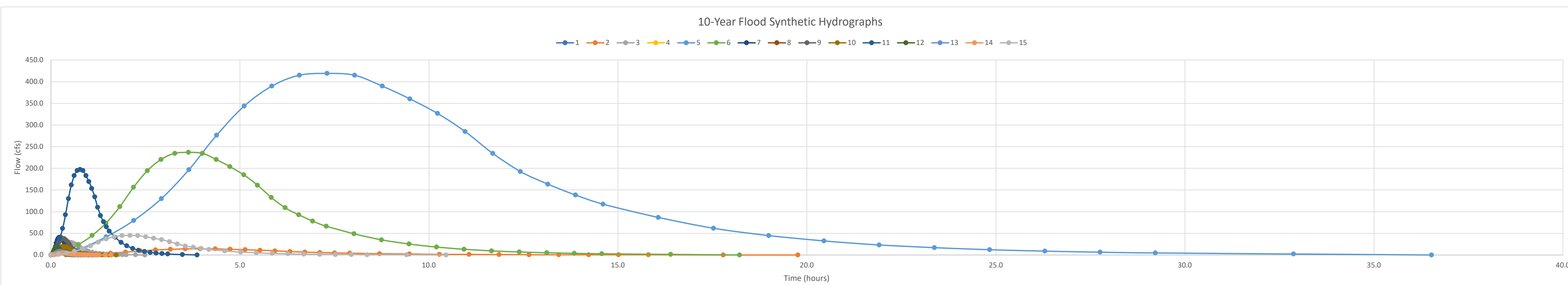
(MMI, 2021)

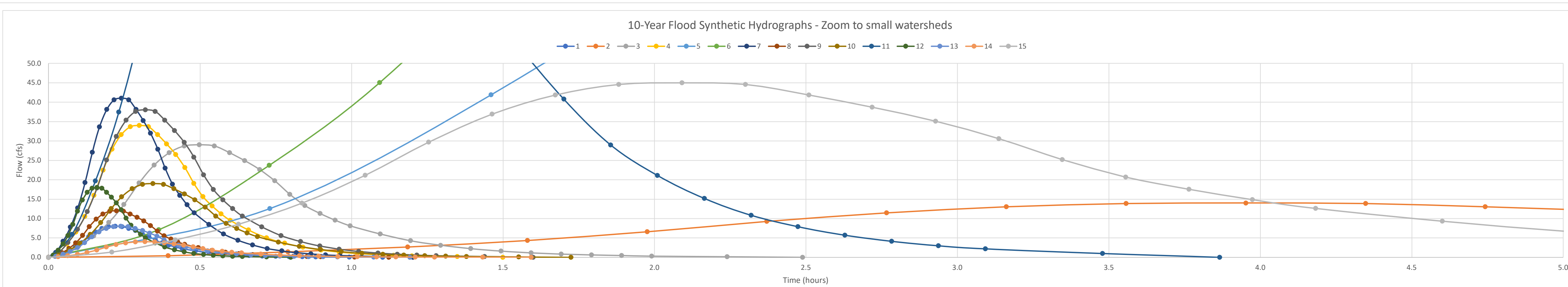
Estimated 10-Year Flood Hydrographs
NRCS Synthetic Hydrographs with USGS StreamStats Peaks and Tc Calculations
4/22/2020

Flow Input ID	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Flow Input Name	Unnamed Tributary	Nilus Brook	Unnamed Tributary	Developed Area	Hampton River	Hampton Falls River	Browns River	Unnamed Tributary	Hunts Island Creek	Unnamed Tributary	Cains Brook Mill Creek	Morrills Brook	Unnamed Tributary	Unnamed Tributary	Blackwater River
Drainage Area (Square Miles)	0.1	0.8	0.4	0.2	15.9	6.9	0.7	0.1	0.3	0.2	2.7	0.2	0.1	0.3	7.4
10-Year Flood Peak (cfs)	8	14	29	34	419	237	41	12	38	19	197	18	8	4	45
Tc (hours)	0.3	5.9	0.7	0.5	11.0	5.5	0.4	0.3	0.5	0.5	1.2	0.2	0.4	0.5	3.1
Lag (hours)	0.2	3.6	0.4	0.3	6.6	3.3	0.2	0.2	0.3	0.3	0.7	0.1	0.2	0.3	1.9
Duration (hours)	0.04	0.79	0.10	0.06	1.46	0.73	0.05	0.04	0.06	0.07	0.15	0.03	0.05	0.06	0.42
Tp (hours)	0.2	4.0	0.5	0.3	7.3	3.6	0.2	0.2	0.3	0.3	0.8	0.2	0.2	0.3	2.1

NRCS Unit Hydrograph		1		2		3		4		5		6		7	
		T	Q	T	Q	T	Q	T	Q	T	Q	T	Q	T	Q
T/Tp	Q/Qp	(hours)	(cfs)	(hours)	(cfs)	(hours)	(cfs)	(hours)	(cfs)	(hours)	(cfs)	(hours)	(cfs)	(hours)	(cfs)
0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.1	0.03	0.0	0.2	0.4	0.4	0.0	0.9	0.0	1.0	0.7	12.6	0.4	7.1	0.0	1.2
0.2	0.1	0.0	0.8	0.8	1.4	0.1	2.9	0.1	3.4	1.5	41.9	0.7	23.7	0.0	4.1
0.3	0.19	0.1	1.5	1.2	2.7	0.1	5.5	0.1	6.5	2.2	79.6	1.1	45.0	0.1	7.8
0.4	0.31	0.1	2.5	1.6	4.3	0.2	9.0	0.1	10.5	2.9	129.9	1.5	73.5	0.1	12.7
0.5	0.47	0.1	3.8	2.0	6.6	0.2	13.6	0.1	16.0	3.7	196.9	1.8	111.4	0.1	19.3
0.6	0.66	0.1	5.3	2.4	9.2	0.3	19.1	0.2	22.4	4.4	276.5	2.2	156.4	0.1	27.1
0.7	0.82	0.2	6.6	2.8	11.5	0.3	23.8	0.2	27.9	5.1	343.6	2.6	194.3	0.2	33.6
0.8	0.93	0.2	7.4	3.2	13.0	0.4	27.0	0.2	31.6	5.8	389.7	2.9	220.4	0.2	38.1
0.9	0.99	0.2	7.9	3.6	13.9	0.4	28.7	0.3	33.7	6.6	414.8	3.3	234.6	0.2	40.6
1	1	0.2	8.0	4.0	14.0	0.5	29.0	0.3	34.0	7.3	419.0	3.6	237.0	0.2	41.0
1.1	0.99	0.2	7.9	4.3	13.9	0.5	28.7	0.3	33.7	8.0	414.8	4.0	234.6	0.3	40.6
1.2	0.93	0.3	7.4	4.7	13.0	0.6	27.0	0.4	31.6	8.8	389.7	4.4	220.4	0.3	38.1
1.3	0.86	0.3	6.9	5.1	12.0	0.6	24.9	0.4	29.2	9.5	360.3	4.7	203.8	0.3	35.3
1.4	0.78	0.3	6.2	5.5	10.9	0.7	22.6	0.4	26.5	10.2	326.8	5.1	184.9	0.3	32.0
1.5	0.68	0.3	5.4	5.9	9.5	0.7	19.7	0.4	23.1	11.0	284.9	5.5	161.2	0.4	27.9
1.6	0.56	0.4	4.5	6.3	7.8	0.8	16.2	0.5	19.0	11.7	234.6	5.8	132.7	0.4	23.0
1.7	0.46	0.4	3.7	6.7	6.4	0.8	13.3	0.5	15.6	12.4	192.7	6.2	109.0	0.4	18.9
1.8	0.39	0.4	3.1	7.1	5.5	0.9	11.3	0.5	13.3	13.1	163.4	6.6	92.4	0.4	16.0
1.9	0.33	0.4	2.6	7.5	4.6	0.9	9.6	0.6	11.2	13.9	138.3	6.9	78.2	0.5	13.5
2	0.28	0.4	2.2	7.9	3.9	1.0	8.1	0.6	9.5	14.6	117.3	7.3	66.4	0.5	11.5
2.2	0.207	0.5	1.7	8.7	2.9	1.1	6.0	0.7	7.0	16.1	86.7	8.0	49.1	0.5	8.5
2.4	0.147	0.5	1.2	9.5	2.1	1.2	4.3	0.7	5.0	17.5	61.6	8.7	34.8	0.6	6.0
2.6	0.107	0.6	0.9	10.3	1.5	1.3	3.1	0.8	3.6	19.0	44.8	9.5	25.4	0.6	4.4
2.8	0.077	0.6	0.6	11.1	1.1	1.4	2.2	0.8	2.6	20.5	32.3	10.2	18.2	0.7	3.2
3	0.055	0.7	0.4	11.9	0.8	1.5	1.6	0.9	1.9	21.9	23.0	10.9	13.0	0.7	2.3
3.2	0.04	0.7	0.3	12.6	0.6	1.6	1.2	1.0	1.4	23.4	16.8	11.7	9.5	0.8	1.6
3.4	0.029	0.8	0.2	13.4	0.4	1.7	0.8	1.0	1.0	24.8	12.2	12.4	6.9	0.8	1.2
3.6	0.021	0.8	0.2	14.2	0.3	1.8	0.6	1.1	0.7	26.3	8.8	13.1	5.0	0.9	0.9
3.8	0.015	0.8	0.1	15.0	0.2	1.9	0.4	1.1	0.5	27.8	6.3	13.8	3.6	0.9	0.6
4	0.011	0.9	0.1	15.8	0.2	2.0	0.3	1.2	0.4	29.2	4.6	14.6	2.6	1.0	0.5
4.5	0.005	1.0	0.0	17.8	0.1	2.2	0.1	1.3	0.2	32.9	2.1	16.4	1.2	1.1	0.2
5	0	1.1	0.0	19.8	0.0	2.5	0.0	1.5	0.0	36.5	0.0	18.2	0.0	1.2	0.0

	8		9		10		11		12		13		14		15	
	T	Q	T	Q	T	Q	T	Q	T	Q	T	Q	T	Q	T	Q
T/Tp	(hours)	(cfs)	(hours)	(cfs)	(hours)	(cfs)	(hours)	(cfs)	(hours)	(cfs)	(hours)	(cfs)	(hours)	(cfs)	(hours)	(cfs)
0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.1	0.0	0.4	0.0	1.1	0.0	0.6	0.1	5.9	0.0	0.5	0.0	0.2	0.0	0.1	0.2	1.4
0.2	0.0	1.2	0.1	3.8	0.1	1.9	0.2	19.7	0.0	1.8	0.0	0.8	0.1	0.4	0.4	4.5
0.3	0.1	2.3	0.1	7.2	0.1	3.6	0.2	37.4	0.0	3.4	0.1	1.5	0.1	0.8	0.6	8.6
0.4	0.1	3.7	0.1	11.8	0.1	5.9	0.3	61.1	0.1	5.6	0.1	2.5	0.1	1.2	0.8	14.0
0.5	0.1	5.6	0.2	17.9	0.2	8.9	0.4	92.6	0.1	8.5	0.1	3.8	0.2	1.9	1.0	21.2
0.6	0.1	7.9	0.2	25.1	0.2	12.5	0.5	130.0	0.1	11.9	0.1	5.3	0.2	2.6	1.3	29.7
0.7	0.2	9.8	0.2	31.2	0.2	15.6	0.5	161.5	0.1	14.8	0.2	6.6	0.2	3.3	1.5	36.9
0.8	0.2	11.2	0.3	35.3	0.3	17.7	0.6	183.2	0.1	16.7	0.2	7.4	0.3	3.7	1.7	41.9
0.9	0.2	11.9	0.3	37.6	0.3	18.8	0.7	195.0	0.1	17.8	0.2	7.9	0.3	4.0	1.9	44.6
1	0.2	12.0	0.3	38.0	0.3	19.0	0.8	197.0	0.2	18.0	0.2	8.0	0.3	4.0	2.1	45.0
1.1	0.2	11.9	0.4	37.6	0.4	18.8	0.9	195.0	0.2	17.8	0.3	7.9	0.4	4.0	2.3	44.6
1.2	0.3	11.2	0.4	35.3	0.4	17.7	0.9	183.2	0.2	16.7	0.3	7.4	0.4	3.7	2.5	41.9
1.3	0.3	10.3	0.4	32.7	0.4	16.3	1.0	169.4	0.2	15.5	0.3	6.9	0.4	3.4	2.7	38.7
1.4	0.3	9.4	0.4	29.6	0.5	14.8	1.1	153.7	0.2	14.0	0.3	6.2	0.4	3.1	2.9	35.1
1.5	0.3	8.2	0.5	25.8	0.5	12.9	1.2	134.0	0.2	12.2	0.4	5.4	0.5	2.7	3.1	30.6
1.6	0.4	6.7	0.5	21.3	0.6	10.6	1.2	110.3	0.3	10.1	0.4	4.5	0.5	2.2	3.3	25.2
1.7	0.4	5.5	0.5	17.5	0.6	8.7	1.3	90.6	0.3	8.3	0.4	3.7	0.5	1.8	3.6	20.7
1.8	0.4	4.7	0.6	14.8	0.6	7.4	1.4	76.8	0.3	7.0	0.4	3.1	0.6	1.6	3.8	17.6
1.9	0.4	4.0	0.6	12.5	0.7	6.3	1.5	65.0	0.3	5.9	0.5	2.6	0.6	1.3	4.0	14.9
2	0.4	3.4	0.6	10.6	0.7	5.3	1.5	55.2	0.3	5.0	0.5	2.2	0.6	1.1	4.2	12.6
2.2	0.5	2.5	0.7	7.9	0.8	3.9	1.7	40.8	0.4	3.7	0.5	1.7	0.7	0.8	4.6	9.3
2.4	0.5	1.8	0.8	5.6	0.8	2.8	1.9	29.0	0.4	2.6	0.6	1.2	0.8	0.6	5.0	6.6
2.6	0.6	1.3	0.8	4.1	0.9	2.0	2.0	21.1	0.4	1.9	0.6	0.9	0.8	0.4	5.4	4.8
2.8	0.6	0.9	0.9	2.9	1.0	1.5	2.2	15.2	0.4	1.4	0.7	0.6	0.9	0.3	5.9	3.5
3	0.7	0.7	1.0	2.1	1.0	1.0	2.3	10.8	0.5	1.0	0.7	0.4	1.0	0.2	6.3	2.5
3.2	0.7	0.5	1.0	1.5	1.1	0.8	2.5	7.9	0.5	0.7	0.8	0.3	1.0	0.2	6.7	1.8
3.4	0.8	0.3	1.1	1.1	1.2	0.6	2.6	5.7	0.5	0.5	0.8	0.2	1.1	0.1	7.1	1.3
3.6	0.8	0.3	1.2	0.8	1.2	0.4	2.8	4.1	0.6	0.4	0.9	0.2	1.1	0.1	7.5	0.9
3.8	0.9	0.2	1.2	0.6	1.3	0.3	2.9	3.0	0.6	0.3	0.9	0.1	1.2	0.1	7.9	0.7
4	0.9	0.1	1.3	0.4	1.4	0.2	3.1	2.2	0.6	0.2	1.0	0.1	1.3	0.0	8.4	0.5
4.5	1.0	0.1	1.4	0.2	1.6	0.1	3.5	1.0	0.7	0.1	1.1	0.0	1.4	0.0	9.4	0.2
5	1.1	0.0	1.6	0.0	1.7	0.0	3.9	0.0	0.8	0.0	1.2	0.0	1.6	0.0	10.5	0.0





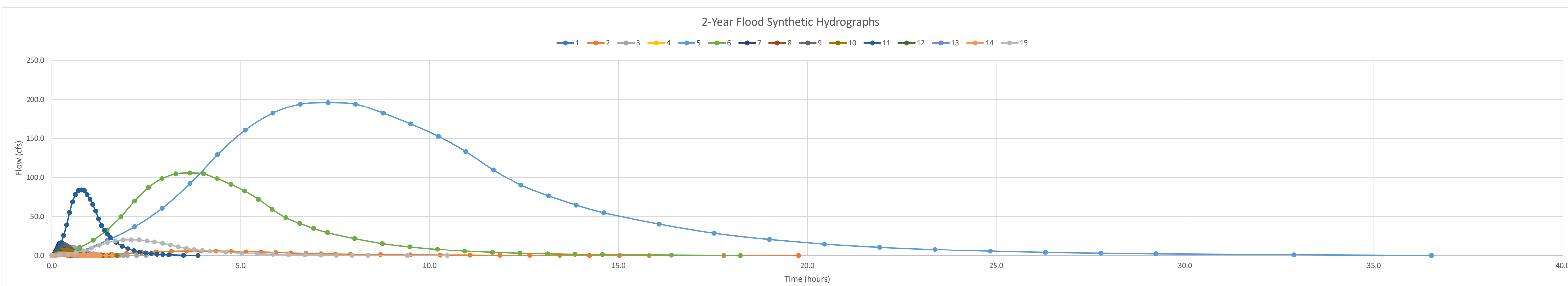
(MMI, 2021)

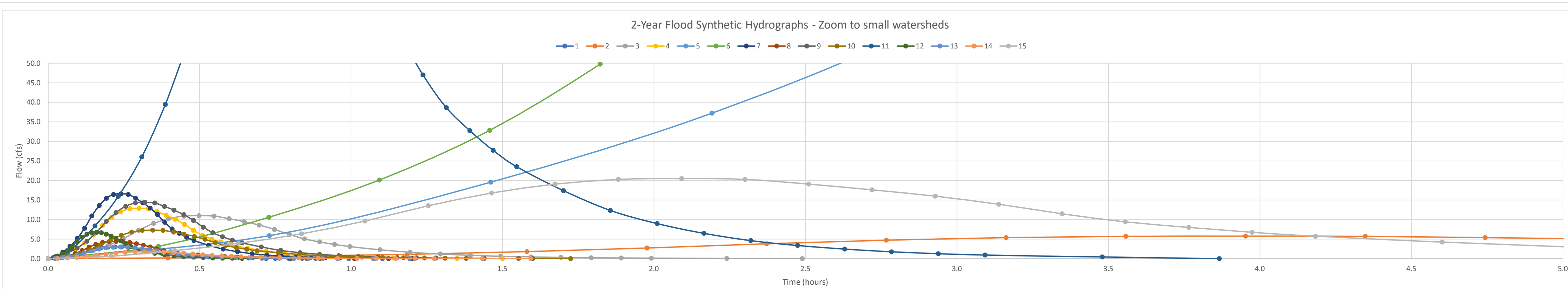
Estimated 2-Year Flood Hydrographs
NRCS Synthetic Hydrographs with USGS StreamStats Peaks and Tc Calculations
4/22/2020

Flow Input ID	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Flow Input Name	Unnamed Tributary	Nilus Brook	Unnamed Tributary	Developed Area	Hampton River	Hampton Falls River	Browns River	Unnamed Tributary	Hunts Island Creek	Unnamed Tributary	Cains Brook Mill Creek	Morrills Brook	Unnamed Tributary	Unnamed Tributary	Blackwater River
Drainage Area (Square Miles)	0.1	0.8	0.4	0.2	15.9	6.9	0.7	0.1	0.3	0.2	2.7	0.2	0.1	0.3	7.4
2-Year Flood Peak (cfs)	3	6	11	13	196	106	17	4	14	7	84	7	3	2	21
Tc (hours)	0.3	5.9	0.7	0.5	11.0	5.5	0.4	0.3	0.5	0.5	1.2	0.2	0.4	0.5	3.1
Lag (hours)	0.2	3.6	0.4	0.3	6.6	3.3	0.2	0.2	0.3	0.3	0.7	0.1	0.2	0.3	1.9
Duration (hours)	0.04	0.79	0.10	0.06	1.46	0.73	0.05	0.04	0.06	0.07	0.15	0.03	0.05	0.06	0.42
Tp (hours)	0.2	4.0	0.5	0.3	7.3	3.6	0.2	0.2	0.3	0.3	0.8	0.2	0.2	0.3	2.1

NRCS Unit Hydrograph		1		2		3		4		5		6		7		8	
		T	Q	T	Q	T	Q	T	Q	T	Q	T	Q	T	Q	T	Q
T/Tp	Q/Qp	(hours)	(cfs)	(hours)	(cfs)	(hours)	(cfs)	(hours)	(cfs)	(hours)	(cfs)	(hours)	(cfs)	(hours)	(cfs)	(hours)	(cfs)
0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.1	0.03	0.0	0.1	0.4	0.2	0.0	0.3	0.0	0.4	0.7	5.9	0.4	3.2	0.0	0.5	0.0	0.1
0.2	0.1	0.0	0.3	0.8	0.6	0.1	1.1	0.1	1.3	1.5	19.6	0.7	10.6	0.0	1.7	0.0	0.4
0.3	0.19	0.1	0.6	1.2	1.1	0.1	2.1	0.1	2.5	2.2	37.2	1.1	20.1	0.1	3.2	0.1	0.8
0.4	0.31	0.1	0.9	1.6	1.8	0.2	3.4	0.1	4.0	2.9	60.8	1.5	32.9	0.1	5.1	0.1	1.4
0.5	0.47	0.1	1.4	2.0	2.7	0.2	5.2	0.1	6.1	3.7	92.1	1.8	49.8	0.1	7.8	0.1	2.1
0.6	0.66	0.1	2.0	2.4	3.8	0.3	7.3	0.2	8.5	4.4	129.4	2.2	70.0	0.1	11.0	0.1	2.9
0.7	0.82	0.2	2.5	2.8	4.7	0.3	9.0	0.2	10.6	5.1	160.7	2.6	86.9	0.2	13.6	0.2	3.6
0.8	0.93	0.2	2.8	3.2	5.4	0.4	10.2	0.2	12.0	5.8	182.3	2.9	98.6	0.2	15.4	0.2	4.1
0.9	0.99	0.2	3.0	3.6	5.7	0.4	10.9	0.3	12.8	6.6	194.0	3.3	104.9	0.2	16.4	0.2	4.4
1	1	0.2	3.0	4.0	5.8	0.5	11.0	0.3	12.9	7.3	196.0	3.6	106.0	0.2	16.6	0.2	4.4
1.1	0.99	0.2	3.0	4.3	5.7	0.5	10.9	0.3	12.8	8.0	194.0	4.0	104.9	0.3	16.4	0.2	4.4
1.2	0.93	0.3	2.8	4.7	5.4	0.6	10.2	0.4	12.0	8.8	182.3	4.4	98.6	0.3	15.4	0.3	4.1
1.3	0.86	0.3	2.6	5.1	5.0	0.6	9.5	0.4	11.1	9.5	168.6	4.7	91.2	0.3	14.3	0.3	3.8
1.4	0.78	0.3	2.4	5.5	4.5	0.7	8.6	0.4	10.1	10.2	152.9	5.1	82.7	0.3	12.9	0.3	3.5
1.5	0.68	0.3	2.1	5.9	3.9	0.7	7.5	0.4	8.8	11.0	133.3	5.5	72.1	0.4	11.3	0.3	3.0
1.6	0.56	0.4	1.7	6.3	3.2	0.8	6.2	0.5	7.2	11.7	109.8	5.8	59.4	0.4	9.3	0.4	2.5
1.7	0.46	0.4	1.4	6.7	2.7	0.8	5.1	0.5	5.9	12.4	90.2	6.2	48.8	0.4	7.6	0.4	2.0
1.8	0.39	0.4	1.2	7.1	2.3	0.9	4.3	0.5	5.0	13.1	76.4	6.6	41.3	0.4	6.5	0.4	1.7
1.9	0.33	0.4	1.0	7.5	1.9	0.9	3.6	0.6	4.3	13.9	64.7	6.9	35.0	0.5	5.5	0.4	1.5
2	0.28	0.4	0.8	7.9	1.6	1.0	3.1	0.6	3.6	14.6	54.9	7.3	29.7	0.5	4.6	0.4	1.2
2.2	0.207	0.5	0.6	8.7	1.2	1.1	2.3	0.7	2.7	16.1	40.6	8.0	21.9	0.5	3.4	0.5	0.9
2.4	0.147	0.5	0.4	9.5	0.9	1.2	1.6	0.7	1.9	17.5	28.8	8.7	15.6	0.6	2.4	0.5	0.7
2.6	0.107	0.6	0.3	10.3	0.6	1.3	1.2	0.8	1.4	19.0	21.0	9.5	11.3	0.6	1.8	0.6	0.5
2.8	0.077	0.6	0.2	11.1	0.4	1.4	0.8	0.8	1.0	20.5	15.1	10.2	8.2	0.7	1.3	0.6	0.3
3	0.055	0.7	0.2	11.9	0.3	1.5	0.6	0.9	0.7	21.9	10.8	10.9	5.8	0.7	0.9	0.7	0.2
3.2	0.04	0.7	0.1	12.6	0.2	1.6	0.4	1.0	0.5	23.4	7.8	11.7	4.2	0.8	0.7	0.7	0.2
3.4	0.029	0.8	0.1	13.4	0.2	1.7	0.3	1.0	0.4	24.8	5.7	12.4	3.1	0.8	0.5	0.8	0.1
3.6	0.021	0.8	0.1	14.2	0.1	1.8	0.2	1.1	0.3	26.3	4.1	13.1	2.2	0.9	0.3	0.8	0.1
3.8	0.015	0.8	0.0	15.0	0.1	1.9	0.2	1.1	0.2	27.8	2.9	13.8	1.6	0.9	0.2	0.9	0.1
4	0.011	0.9	0.0	15.8	0.1	2.0	0.1	1.2	0.1	29.2	2.2	14.6	1.2	1.0	0.2	0.9	0.0
4.5	0.005	1.0	0.0	17.8	0.0	2.2	0.1	1.3	0.1	32.9	1.0	16.4	0.5	1.1	0.1	1.0	0.0
5	0	1.1	0.0	19.8	0.0	2.5	0.0	1.5	0.0	36.5	0.0	18.2	0.0	1.2	0.0	1.1	0.0

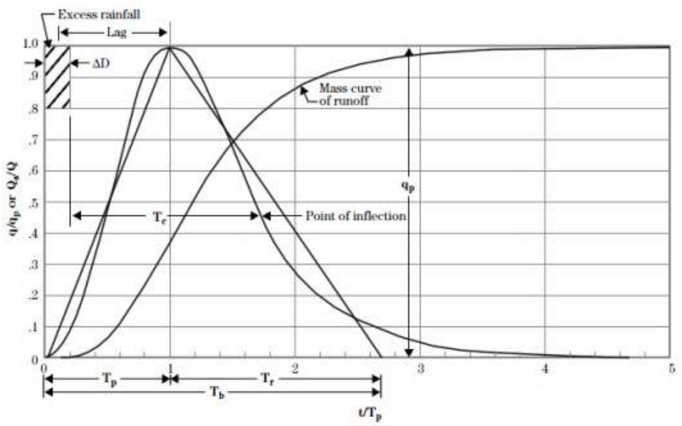
NRCS Unit Hydrograph		9		10		11		12		13		14		15	
		T	Q	T	Q	T	Q	T	Q	T	Q	T	Q	T	Q
T/Tp	Q/Qp	(hours)	(cfs)	(hours)	(cfs)	(hours)	(cfs)	(hours)	(cfs)	(hours)	(cfs)	(hours)	(cfs)	(hours)	(cfs)
0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.1	0.03	0.0	0.4	0.0	0.2	0.1	2.5	0.0	0.2	0.0	0.1	0.0	0.1	0.2	0.6
0.2	0.1	0.1	1.4	0.1	0.7	0.2	8.4	0.0	0.7	0.0	0.3	0.1	0.2	0.4	2.1
0.3	0.19	0.1	2.7	0.1	1.4	0.2	16.0	0.0	1.3	0.1	0.6	0.1	0.3	0.6	3.9
0.4	0.31	0.1	4.5	0.1	2.3	0.3	26.0	0.1	2.1	0.1	0.9	0.1	0.5	0.8	6.4
0.5	0.47	0.2	6.8	0.2	3.4	0.4	39.5	0.1	3.2	0.1	1.4	0.2	0.8	1.0	9.6
0.6	0.66	0.2	9.5	0.2	4.8	0.5	55.4	0.1	4.4	0.1	2.0	0.2	1.2	1.3	13.5
0.7	0.82	0.2	11.8	0.2	6.0	0.5	68.9	0.1	5.5	0.2	2.4	0.2	1.4	1.5	16.8
0.8	0.93	0.3	13.4	0.3	6.8	0.6	78.1	0.1	6.2	0.2	2.8	0.3	1.6	1.7	19.1
0.9	0.99	0.3	14.3	0.3	7.2	0.7	83.2	0.1	6.7	0.2	2.9	0.3	1.7	1.9	20.3
1	1	0.3	14.4	0.3	7.3	0.8	84.0	0.2	6.7	0.2	3.0	0.3	1.8	2.1	20.5
1.1	0.99	0.4	14.3	0.4	7.2	0.9	83.2	0.2	6.7	0.3	2.9	0.4	1.7	2.3	20.3
1.2	0.93	0.4	13.4	0.4	6.8	0.9	78.1	0.2	6.2	0.3	2.8	0.4	1.6	2.5	19.1
1.3	0.86	0.4	12.4	0.4	6.3	1.0	72.2	0.2	5.8	0.3	2.6	0.4	1.5	2.7	17.6
1.4	0.78	0.4	11.2	0.5	5.7	1.1	65.5	0.2	5.2	0.3	2.3	0.4	1.4	2.9	16.0
1.5	0.68	0.5	9.8	0.5	5.0	1.2	57.1	0.2	4.6	0.4	2.0	0.5	1.2	3.1	13.9
1.6	0.56	0.5	8.1	0.6	4.1	1.2	47.0	0.3	3.8	0.4	1.7	0.5	1.0	3.3	11.5
1.7	0.46	0.5	6.6	0.6	3.4	1.3	38.6	0.3	3.1	0.4	1.4	0.5	0.8	3.6	9.4
1.8	0.39	0.6	5.6	0.6	2.8	1.4	32.8	0.3	2.6	0.4	1.2	0.6	0.7	3.8	8.0
1.9	0.33	0.6	4.8	0.7	2.4	1.5	27.7	0.3	2.2	0.5	1.0	0.6	0.6	4.0	6.8
2	0.28	0.6	4.0	0.7	2.0	1.5	23.5	0.3	1.9	0.5	0.8	0.6	0.5	4.2	5.7
2.2	0.207	0.7	3.0	0.8	1.5	1.7	17.4	0.4	1.4	0.5	0.6	0.7	0.4	4.6	4.2
2.4	0.147	0.8	2.1	0.8	1.1	1.9	12.3	0.4	1.0	0.6	0.4	0.8	0.3	5.0	3.0
2.6	0.107	0.8	1.5	0.9	0.8	2.0	9.0	0.4	0.7	0.6	0.3	0.8	0.2	5.4	2.2
2.8	0.077	0.9	1.1	1.0	0.6	2.2	6.5	0.4	0.5	0.7	0.2	0.9	0.1	5.9	1.6
3	0.055	1.0	0.8	1.0	0.4	2.3	4.6	0.5	0.4	0.7	0.2	1.0	0.1	6.3	1.1
3.2	0.04	1.0	0.6	1.1	0.3	2.5	3.4	0.5	0.3	0.8	0.1	1.0	0.1	6.7	0.8
3.4	0.029	1.1	0.4	1.2	0.2	2.6	2.4	0.5	0.2	0.8	0.1	1.1	0.1	7.1	0.6
3.6	0.021	1.2	0.3	1.2	0.2	2.8	1.8	0.6	0.1	0.9	0.1	1.1	0.0	7.5	0.4
3.8	0.015	1.2	0.2	1.3	0.1	2.9	1.3	0.6	0.1	0.9	0.0	1.2	0.0	7.9	0.3
4	0.011	1.3	0.2	1.4	0.1	3.1	0.9	0.6	0.1	1.0	0.0	1.3	0.0	8.4	0.2
4.5	0.005	1.4	0.1	1.6	0.0	3.5	0.4	0.7	0.0	1.1	0.0	1.4	0.0	9.4	0.1
5	0	1.6	0.0	1.7	0.0	3.9	0.0	0.8	0.0	1.2	0.0	1.6	0.0	10.5	0.0





(MMI, 2021)

METHODS



$$q_p = \frac{484AQ}{T_p}$$

http://www.professorpatel.com/scs-dimensionless-unit-hydrograph.html
NRCS Engieering Handbook

A is the drainage area in square miles
Q is the runoff volume in inches
T_p is the time to peak in hours, and
q_p is the peak flow rate in cfs.

The peak rate factor of 484 has the inherent assumption that 3/8 of the volume under the unit hydrograph is under the rising limb and the remaining 5/8 of the volume is under the recession limb. This may not be true if the study area h

To use the SCS DUH, we need to determine only two things:
1. Time to peak, T_p (hr), and
2. Peak discharge, q_p (cfs).

The time to peak can be determined as follows:
$T_p=(D/2)+T_l$,
where,
T_l is the lag time (hr) and
D = duration of the rainfall (hr),
The DUH has point of inflection located at approximately $1.7T_p$. So, using our relation of $T_l=0.6*T_c$, we can compute D as:
$D = 0.2*T_p$ or $D = 0.133*T_c$.
Small variation in D is ok, but it should not exceed $0.25T_p$ or $0.17T_c$.

The peak discharge can be determined as follows:
$q_p=(484*A)/T_p$, which is same as the equation shown previously, but with Q = 1.0 inch for the unit hydrograph.
If you need to determine the discharge for any other runoff volume, you can multiply the q_p with appropriate runoff depth, Q (in).
Once we determine T_p and q_p, we can calculate D-hr unit hydrograph for our drainage area of interest using following co-ordinates:

t/t_p	q/q_p	t/t_p	q/q_p
0	0	1.70	0.46
0.10	0.03	1.80	0.39
0.20	0.1	1.90	0.33
0.30	0.19	2.00	0.28
0.40	0.31	2.20	0.207
0.50	0.47	2.40	0.147
0.60	0.66	2.60	0.107
0.70	0.82	2.80	0.077
0.80	0.93	3.00	0.055
0.90	0.99	3.20	0.04
1.00	1.00	3.40	0.029
1.10	0.99	3.60	0.021
1.20	0.93	3.80	0.015
1.30	0.86	4.00	0.011
1.40	0.78	4.50	0.005
1.50	0.68	5.00	0
1.60	0.56		

Using figure 10, $T_p= \frac{D}{2}+$ L and L = 0.6 T_c were developed from analysis of small watershed data.

D Equation

From Empirical Relations:
(This equation was developed from small watershed data)

Lag = 0.6 T_c

Point of inflection - 1.7 T_c

T_p= D/2 + Lag

From The Dimensionless Unit Hydrograph:

(1) T_c+ D = 1.7 T_p

(2) D /2 + 0.6 T_c= T_p

Solving Equations (1) And (2)

T_c+ D = 1. 7 (D /2 + 0.6 T_c)

T_c+ D = 0.85 D + 1.02 T_c

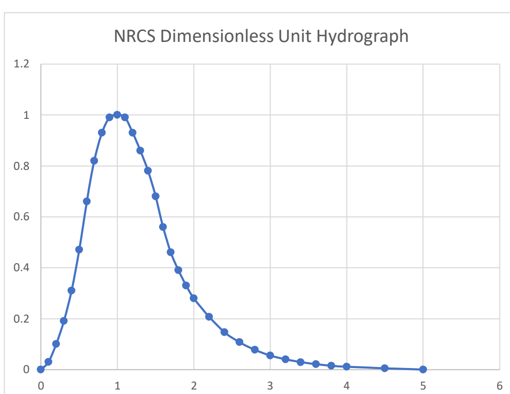
0.15 D = 0.02 T_c

D = 0.133 T_c

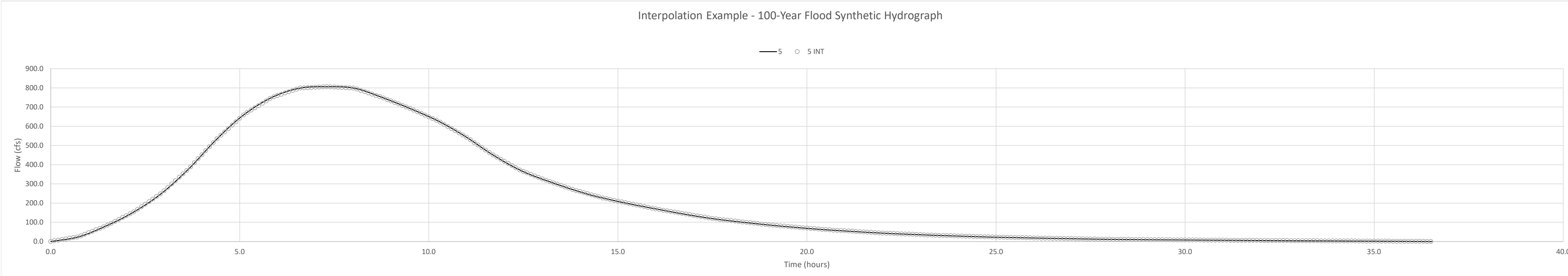
Figure 14. Derivation of equations for D.

NRCS Dimensionless Unit Hydrograph

t/tp	q/qp
0	0
0.1	0.03
0.2	0.1
0.3	0.19
0.4	0.31
0.5	0.47
0.6	0.66
0.7	0.82
0.8	0.93
0.9	0.99
1	1
1.1	0.99
1.2	0.93
1.3	0.86
1.4	0.78
1.5	0.68
1.6	0.56
1.7	0.46
1.8	0.39
1.9	0.33
2	0.28
2.2	0.207
2.4	0.147
2.6	0.107
2.8	0.077
3	0.055
3.2	0.04
3.4	0.029
3.6	0.021
3.8	0.015
4	0.011
4.5	0.005
5	0



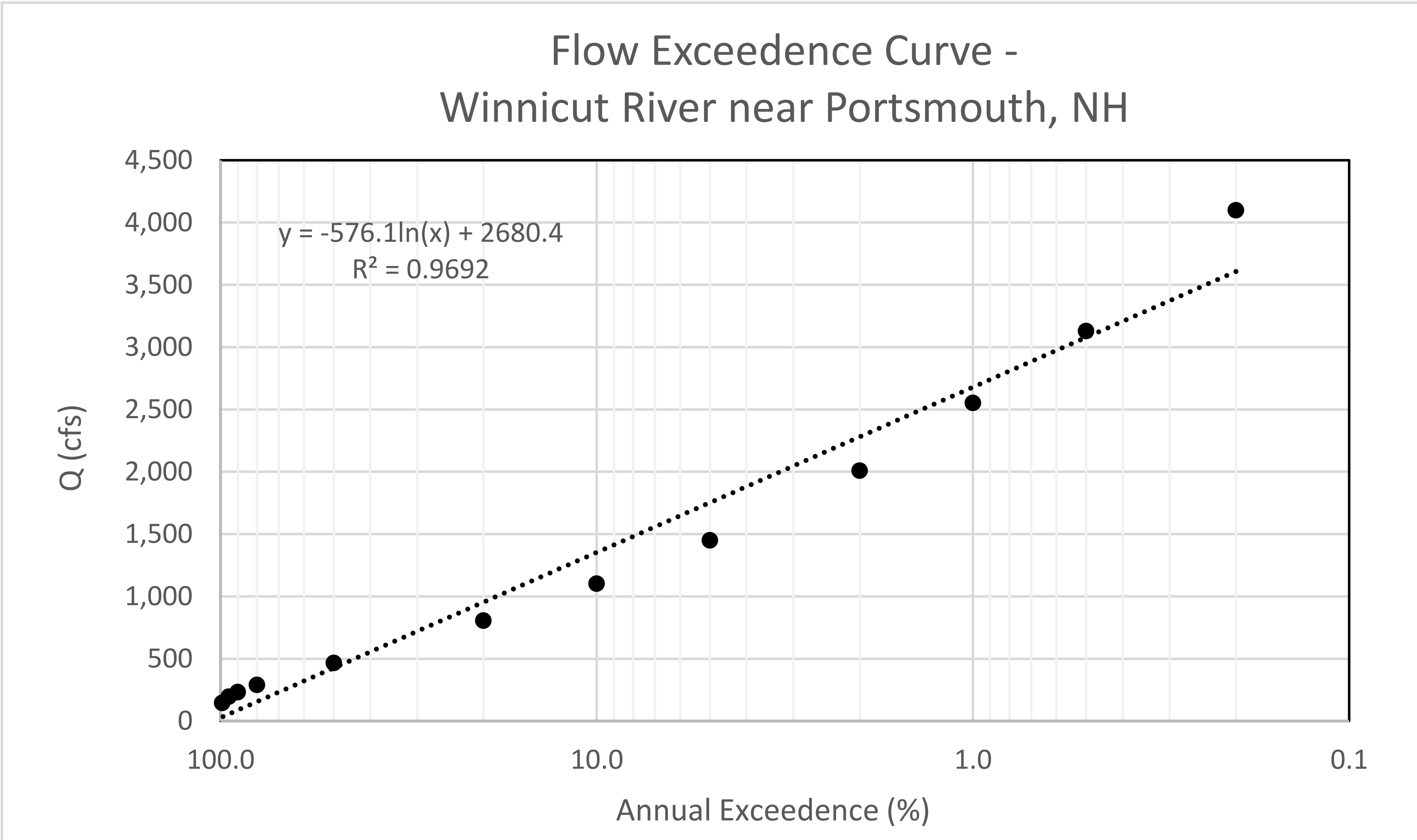
(MMI, 2021)



INTERPOLATED VALUES FOR BOUNDARY CONDITIONS

1 INT		2 INT		3 INT		4 INT		5 INT		6 INT		7 INT		8 INT		9 INT		10 INT		11 INT		12 INT		13 INT		14 INT		15 INT	
T_int (hours)	Q_int (cfs)	T_int (hours)	Q_int (cfs)	T_int (hours)	Q_int (cfs)	T_int (hours)	Q_int (cfs)	T_int (hours)	Q_int (cfs)	T_int (hours)	Q_int (cfs)	T_int (hours)	Q_int (cfs)	T_int (hours)	Q_int (cfs)	T_int (hours)	Q_int (cfs)	T_int (hours)	Q_int (cfs)	T_int (hours)	Q_int (cfs)	T_int (hours)	Q_int (cfs)	T_int (hours)	Q_int (cfs)	T_int (hours)	Q_int (cfs)	T_int (hours)	Q_int (cfs)
0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0
0.1	7.9	0.1	0.2	0.1	6.3	0.1	18.2	0.1	3.3	0.1	3.8	0.1	30.1	0.1	11.1	0.1	17.6	0.1	8.3	0.1	20.8	0.1	30.1	0.1	6.5	0.1	2.1	0.1	1.2
0.2	19.8	0.2	0.5	0.2	19.7	0.2	60.6	0.2	6.6	0.2	7.7	0.2	85.4	0.2	28.5	0.2	60.2	0.2	28.6	0.2	63.0	0.2	38.5	0.2	18.1	0.2	7.0	0.2	2.5
0.3	16.2	0.3	0.7	0.3	41.8	0.3	79.0	0.3	9.9	0.3	11.5	0.3	80.7	0.3	24.1	0.3	85.5	0.3	44.7	0.3	121.8	0.3	14.8	0.3	16.9	0.3	9.9	0.3	5.2
0.4	7.6	0.4	0.9	0.4	58.7	0.4	65.8	0.4	13.2	0.4	17.2	0.4	44.8	0.4	11.7	0.4	77.0	0.4	43.9	0.4	207.2	0.4	5.5	0.4	9.2	0.4	8.9	0.4	8.1
0.5	3.7	0.5	1.5	0.5	63.0	0.5	38.9	0.5	16.6	0.5	26.1	0.5	22.6	0.5	5.8	0.5	52.0	0.5	33.6	0.5	302.7	0.5	2.0	0.5	4.7	0.5	6.0	0.5	11.6
0.6	1.8	0.6	2.0	0.6	58.4	0.6	22.1	0.6	19.9	0.6	35.1	0.6	11.5	0.6	2.8	0.6	29.6	0.6	19.9	0.6	372.3	0.6	0.7	0.6	2.4	0.6	3.4	0.6	15.3
0.7	0.8	0.7	2.5	0.7	48.8	0.7	13.2	0.7	23.2	0.7	44.0	0.7	5.8	0.7	1.3	0.7	18.2	0.7	12.4	0.7	408.1	0.7	0.3	0.7	1.2	0.7	2.1	0.7	19.9
0.8	0.4	0.8	3.1	0.8	34.8	0.8	7.7	0.8	29.5	0.8	54.8	0.8	3.0	0.8	0.7	0.8	10.9	0.8	7.9	0.8	410.6	0.8	0.0	0.8	0.6	0.8	1.2	0.8	24.9
0.9	0.2	0.9	3.7	0.9	24.3	0.9	4.3	0.9	37.3	0.9	66.3	0.9	1.5	0.9	0.3	0.9	6.5	0.9	4.9	0.9	392.0			0.9	0.3	0.9	0.7	0.9	30.8
1	0.1	1	4.4	1	17.4	1	2.6	1	45.0	1	77.8	1	0.8	1	0.2	1	3.9	1	3.0	1	356.2			1	0.2	1	0.4	1	37.4
1.1	0.0	1.1	5.1	1.1	12.9	1.1	1.5	1.1	52.7	1.1	89.6	1.1	0.4	1.1	0.0	1.1	2.4	1.1	1.9	1.1	312.0			1.1	0.1	1.1	0.3	1.1	44.7
		1.2	5.8	1.2	9.1	1.2	0.9	1.2	60.4	1.2	104.9	1.2	0.0			1.2	1.4	1.2	1.2	1.2	254.4			1.2	0.0	1.2	0.2	1.2	52.5
		1.3	6.7	1.3	6.6	1.3	0.6	1.3	68.2	1.3	120.3					1.3	0.9	1.3	0.7	1.3	197.2					1.3	0.1	1.3	59.7
		1.4	7.7	1.4	4.8	1.4	0.3	1.4	75.9	1.4	135.6					1.4	0.6	1.4	0.5	1.4	158.0					1.4	0.1	1.4	66.3
		1.5	8.6	1.5	3.4	1.5	0.0	1.5	84.5	1.5	153.2					1.5	0.3	1.5	0.3	1.5	127.7					1.5	0.0	1.5	72.1
		1.6	9.5	1.6	2.5			1.6	94.4	1.6	173.6					1.6	0.0	1.6	0.2	1.6	104.9					1.6	0.0	1.6	76.7
		1.7	10.7	1.7	1.8			1.7	104.3	1.7	194.1							1.7	0.0	1.7	85.5							1.7	80.6
		1.8	12.0	1.8	1.3			1.8	114.3	1.8	214.5									1.8	69.4							1.8	83.1
		1.9	13.2	1.9	0.9			1.9	124.2	1.9	238.0									1.9	55.8							1.9	85.2
		2	14.4	2	0.7			2	134.1	2	262.3									2	45.2							2	85.6
		2.1	15.9	2.1	0.5			2.1	144.1	2.1	286.6									2.1	36.9							2.1	86.0
		2.2	17.3	2.2	0.4			2.2	154.3	2.2	310.4									2.2	29.7							2.2	85.6
		2.3	18.8	2.3	0.2			2.3	167.5	2.3	330.8									2.3	23.8							2.3	85.1
		2.4	20.1	2.4	0.1			2.4	180.8	2.4	351.3									2.4	19.4							2.4	82.7
		2.5	21.4	2.5	0.0			2.5	194.0	2.5	371.8									2.5	15.7							2.5	80.2
		2.6	22.6					2.6	207.2	2.6	389.1									2.6	12.8							2.6	77.4
		2.7	23.8					2.7	220.5	2.7	403.1									2.7	10.4							2.7	74.5
		2.8	24.9					2.8	233.7	2.8	417.2									2.8	8.4							2.8	71.3
		2.9	25.7					2.9	247.0	2.9	431.3									2.9	6.8							2.9	68.0
		3	26.5					3	263.6	3	439.9									3	5.5							3	64.1
		3.1	27.4					3.1	281.3	3.1	447.6									3.1	4.5							3.1	60.0
		3.2	28.1					3.2	298.9	3.2	455.2									3.2	3.8							3.2	55.4
		3.3	28.5					3.3	316.6	3.3	461.6									3.3	3.2							3.3	50.4
		3.4	29.0					3.4	334.3	3.4	462.9									3.4	2.6							3.4	46.0
		3.5	29.4					3.5	351.9	3.5	464.2									3.5	1.9							3.5	41.8
		3.6	29.7					3.6	369.6	3.6	465.4									3.6	1.4							3.6	38.3
		3.7	29.8					3.7	388.8	3.7	465.3									3.7	0.9							3.7	35.4
		3.8	29.9					3.8	409.8	3.8	464.0									3.8	0.4							3.8	32.7
		3.9	30.0					3.9	430.7	3.9	462.7									3.9	0.0							3.9	30.2
		4	30.0					4	451.7	4	461.4																	4	27.8
		4.1	29.9					4.1	472.7	4.1	454.3																	4.1	25.8
		4.2	29.8					4.2	493.6	4.2	446.6																	4.2	23.8
		4.3	29.7					4.3	514.6	4.3	438.9																	4.3	22.3
		4.4	29.5					4.4	535.0	4.4	430.9																	4.4	20.8
		4.5	29.0					4.5	552.6	4.5	422.0																	4.5	19.3
		4.6	28.6					4.6	570.3	4.6	413.0																	4.6	17.8
		4.7	28.1					4.7	587.9	4.7	404.1																	4.7	16.6
		4.8	27.6					4.8	605.6	4.8	394.3																	4.8	15.4
		4.9	27.1					4.9	623.2	4.9	384.1																	4.9	14.1
		5	26.5					5	640.9	5	373.8																	5	12.9
		5.1	26.0					5.1	658.6	5.1	363.6																	5.1	12.0
		5.2	25.4					5.2	671.4	5.2	350.9																	5.2	11.2
		5.3	24.8					5.3	683.6	5.3	338.1																	5.3	10.3
		5.4	24.2					5.4	695.7	5.4	325.3																	5.4	9.5
		5.5	23.6					5.5	707.8	5.5	311.6																	5.5	8.8
		5.6	22.9					5.6	720.0	5.6	296.3																	5.6	8.2
		5.7	22.1					5.7	732.1	5.7	280.9																	5.7	7.6
		5.8	21.4					5.8	744.3	5.8	265.6																	5.8	7.0
		5.9	20.6					5.9	753.3	5.9	252.0																	5.9	6.4
		6	19.7					6	759.9	6	239.2																	6	6.0
		6.1	18.8					6.1	766.5	6.1	226.4																	6.1	5.5
		6.2	17.9					6.2	773.2	6.2	213.9																	6.2	5.1
		6.3	17.0					6.3	779.8	6.3	204.9																	6.3	4.7

(MMI, 2021)



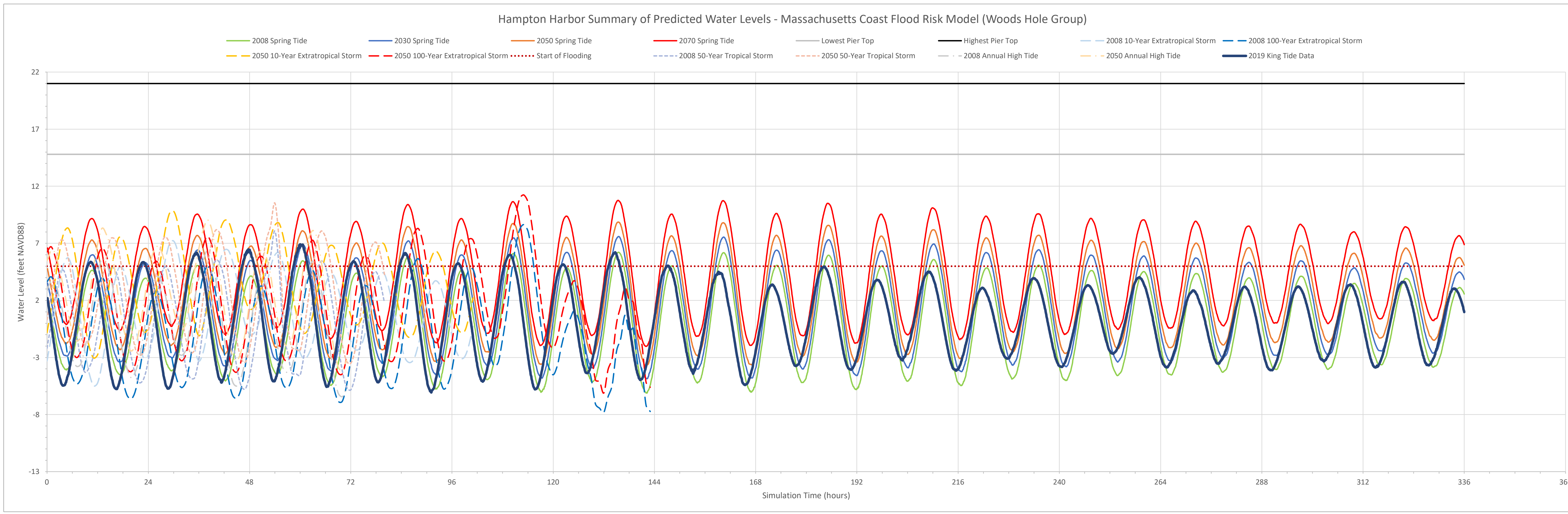
Flow Exceedence Curve -
Winnicut River near Portsmouth, NH
4,500
4,000
3,500
3,000
2,500
2,000
1,500
1,000
500
0
Q (cfs)
y = -576.1ln(x) + 2680.4
R² = 0.9692
100.0
10.0
1.0
0.1
Annual Exceedence (%)

(MMI, 2021)

Peak Flow Scaling From Winnicut River to Fifteen (15) Inflow Locations

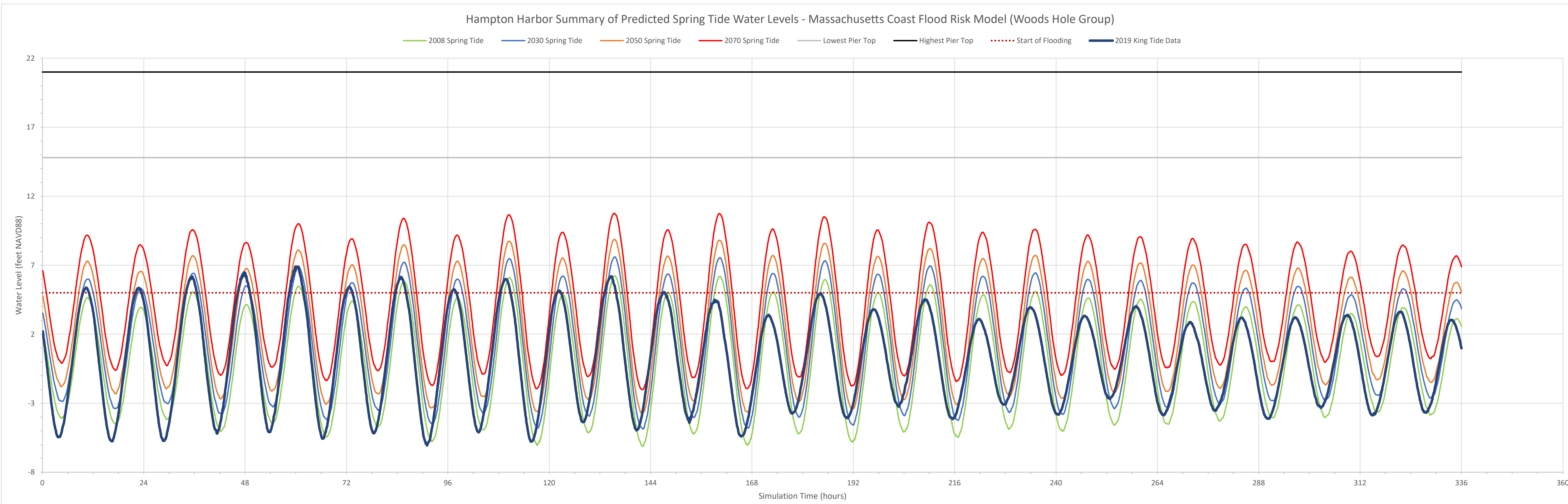
Flow Input ID	Flow Input Name	Drainage Area (Square Miles)	Site Area Ratio (m=0.75)	Drainage Area Difference (%)	2 Year Peak Flood (cfs)	5 Year Peak Flood	10 Year Peak Flood	50 Year Peak Flood	100 Year Peak Flood	500 Year Peak Flood
1	Unnamed Tributary	0.1	0.02	-99%	10	17	23	42	53	85
2	Nilus Brook	0.8	0.11	-94%	53	92	126	229	291	467
3	Unnamed Tributary	0.4	0.07	-97%	30	52	72	131	166	267
4	Developed Area	0.2	0.05	-98%	22	38	52	95	120	193
5	Hampton River	15.9	1.09	13%	510	880	1,206	2,199	2,793	4,484
6	Hampton Falls River	6.9	0.58	-51%	272	470	644	1,174	1,492	2,395
7	Browns River	0.7	0.10	-95%	48	83	113	207	263	422
8	Unnamed Tributary	0.1	0.03	-99%	13	23	31	56	72	115
9	Hunts Island Creek	0.3	0.06	-98%	26	45	61	112	142	228
10	Unnamed Tributary	0.2	0.04	-99%	18	32	44	79	101	162
11	Cains Brook Mill Creek	2.7	0.29	-81%	135	233	319	582	739	1,186
12	Morrills Brook	0.2	0.03	-99%	15	27	36	67	85	136
13	Unnamed Tributary	0.1	0.03	-99%	13	23	31	56	72	115
14	Unnamed Tributary	0.3	0.06	-98%	29	49	67	123	156	251
15	Blackwater River	7.4	0.61	-48%	286	494	677	1,234	1,567	2,516

(MMI, 2021)



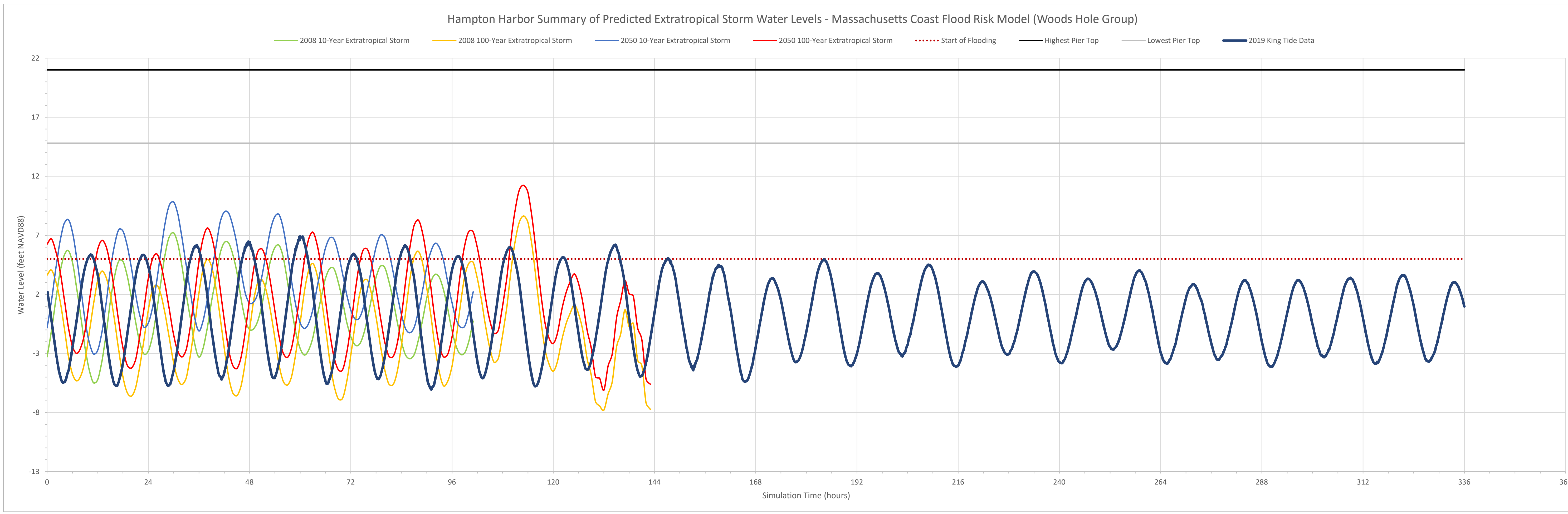
Hampton Harbor Summary of Predicted Water Levels - Massachusetts Coast Flood Risk Model (Woods Hole Group)
2008 Spring Tide
2030 Spring Tide
2050 Spring Tide
2070 Spring Tide
Lowest Pier Top
Highest Pier Top
2008 10-Year Extratropical Storm
2008 100-Year Extratropical Storm
2050 10-Year Extratropical Storm
2050 100-Year Extratropical Storm
Start of Flooding
2008 50-Year Tropical Storm
2050 50-Year Tropical Storm
2008 Annual High Tide
2050 Annual High Tide
2019 King Tide Data
Water Level (feet NAVD88)
22
17
12
7
2
-3
-8
-13
0
24
48
72
96
120
144
168
192
216
240
264
288
312
336
360
Simulation Time (hours)

(MMI, 2021)

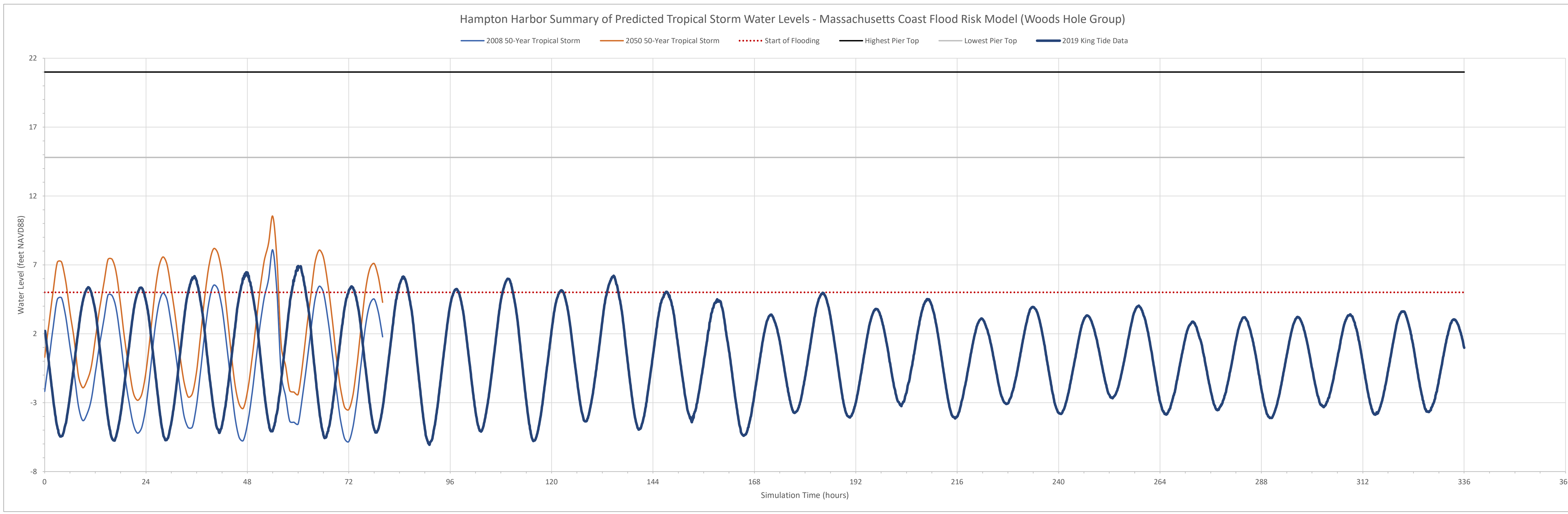


Hampton Harbor Summary of Predicted Spring Tide Water Levels - Massachusetts Coast Flood Risk Model (Woods Hole Group)
2008 Spring Tide
2030 Spring Tide
2050 Spring Tide
2070 Spring Tide
Lowest Pier Top
Highest Pier Top
Start of Flooding
2019 King Tide Data
Water Level (feet NAVD88)
22
17
12
7
2
-3
-8
0
24
48
72
96
120
144
168
192
216
240
264
288
312
336
360
Simulation Time (hours)

(MMI, 2021)

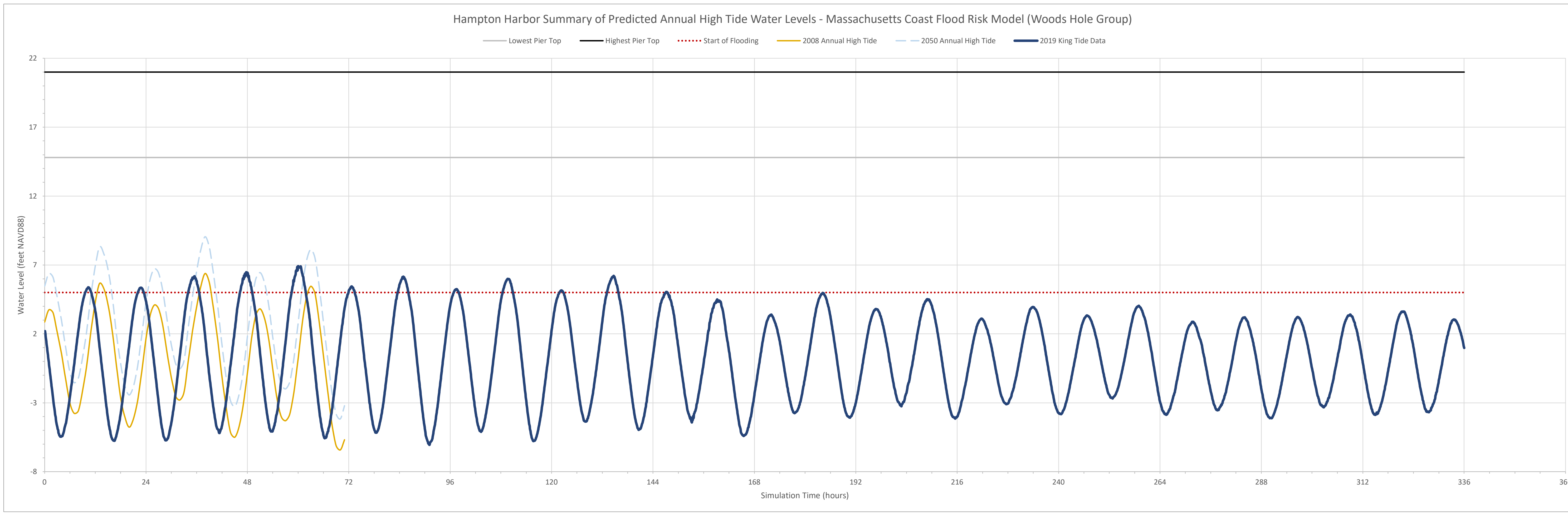

Hampton Harbor Summary of Predicted Extratropical Storm Water Levels - Massachusetts Coast Flood Risk Model (Woods Hole Group)
2008 10-Year Extratropical Storm
2008 100-Year Extratropical Storm
2050 10-Year Extratropical Storm
2050 100-Year Extratropical Storm
Start of Flooding
Highest Pier Top
Lowest Pier Top
2019 King Tide Data
Water Level (feet NAVD88)
22
17
12
7
2
-3
-8
-13
0
24
48
72
96
120
144
168
192
216
240
264
288
312
336
360
Simulation Time (hours)

(MMI, 2021)



Hampton Harbor Summary of Predicted Tropical Storm Water Levels - Massachusetts Coast Flood Risk Model (Woods Hole Group)
2008 50-Year Tropical Storm
2050 50-Year Tropical Storm
Start of Flooding
Highest Pier Top
Lowest Pier Top
2019 King Tide Data
Water Level (feet NAVD88)
22
17
12
7
2
-3
-8
0
24
48
72
96
120
144
168
192
216
240
264
288
312
336
360
Simulation Time (hours)

(MMI, 2021)



Hampton Harbor Summary of Predicted Annual High Tide Water Levels - Massachusetts Coast Flood Risk Model (Woods Hole Group)
Lowest Pier Top
Highest Pier Top
Start of Flooding
2008 Annual High Tide
2050 Annual High Tide
2019 King Tide Data
Water Level (feet NAVD88)
22
17
12
7
2
-3
-8
0
24
48
72
96
120
144
168
192
216
240
264
288
312
336
360
Simulation Time (hours)

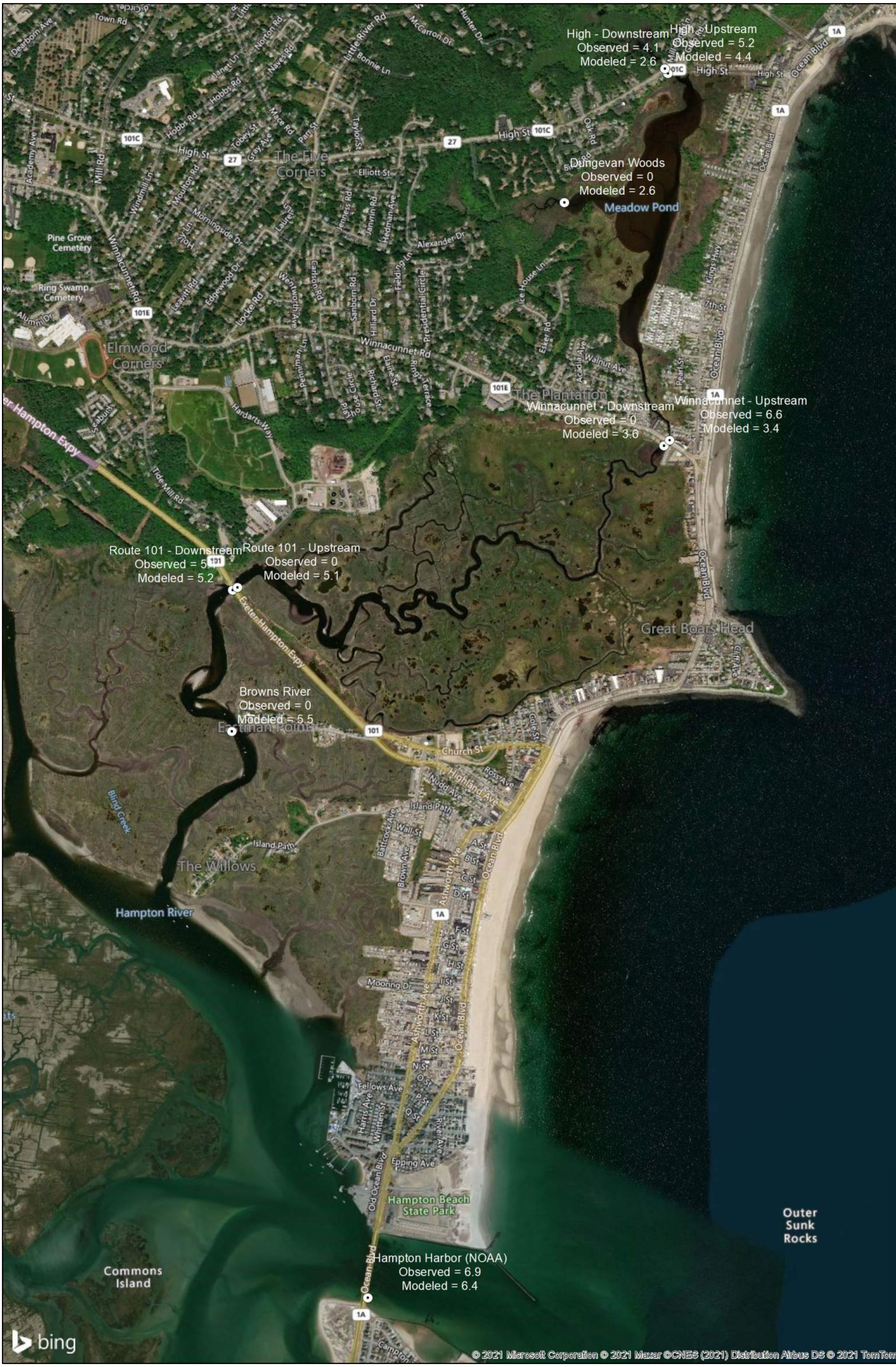
APPENDIX E

EXISTING CONDITIONS HYDRAULICS

Engineering Report

March 2021

Hoyle, Tanner & Associates, Inc.



VALIDATION - King Tide, 10/28/2020, 12:00 PM

HAMPTON FLOOD MITIGATION STUDY
TOWN OF HAMPTON, NEW HAMPSHIRE
(MMI, 2021)

0 237.5 475 950 1,425 1,900
Feet

1 in = 1,200 feet

N

MILONE & MACBROOM — Now part of SLR

1 South Main St
Waterbury, VT 05676
802-882-8335

HYDRAULIC MODEL PRELIMINARY RESULTS - 2019 KING TIDE

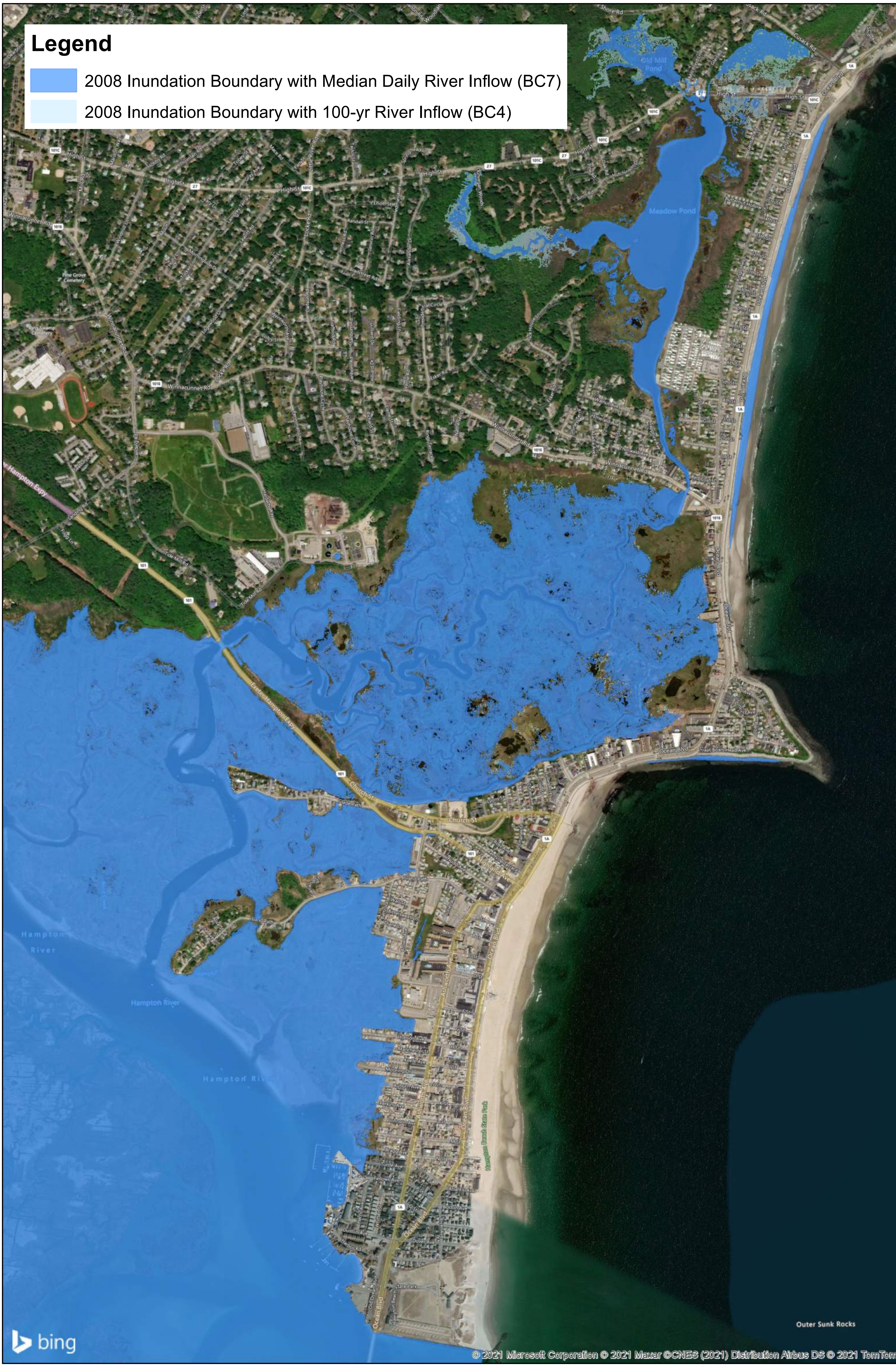
HAMPTON FLOOD STUDY
TOWN OF HAMPTON, NH
(MMI, 2021)

0 250 500 1,000 1,500 2,000
Feet

1 in = 1,250 feet

N

MILONE & MACBROOM
1 South Main St
Waterbury, VT 05676
802-882-8335



HYDRAULIC MODEL RESULTS - PRESENT TIDES

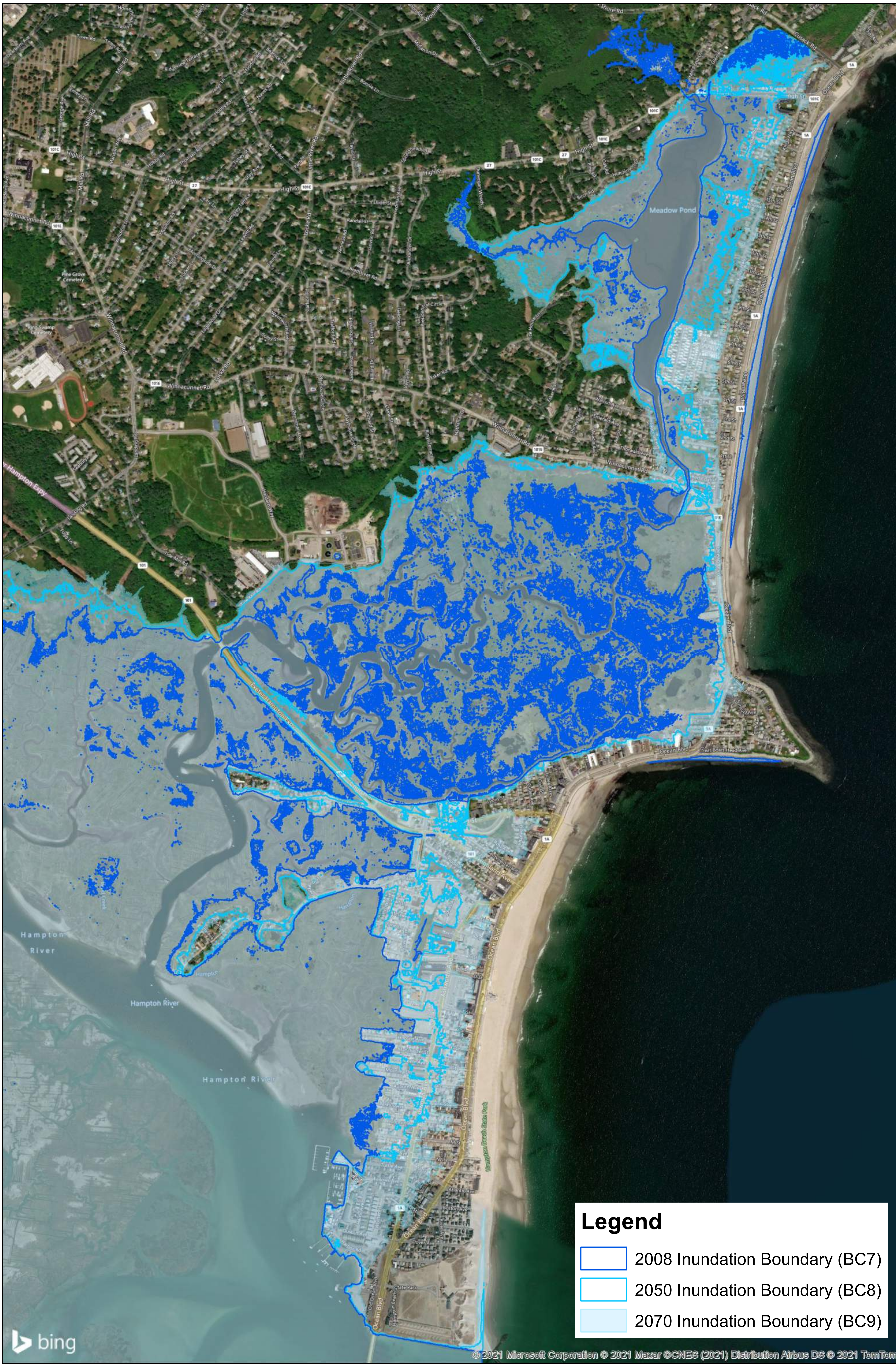
HAMPTON FLOOD STUDY
TOWN OF HAMPTON, NH
(MMI, 2021)

0 225 450 900 1,350 1,800
Feet

1 in = 1,125 feet

N

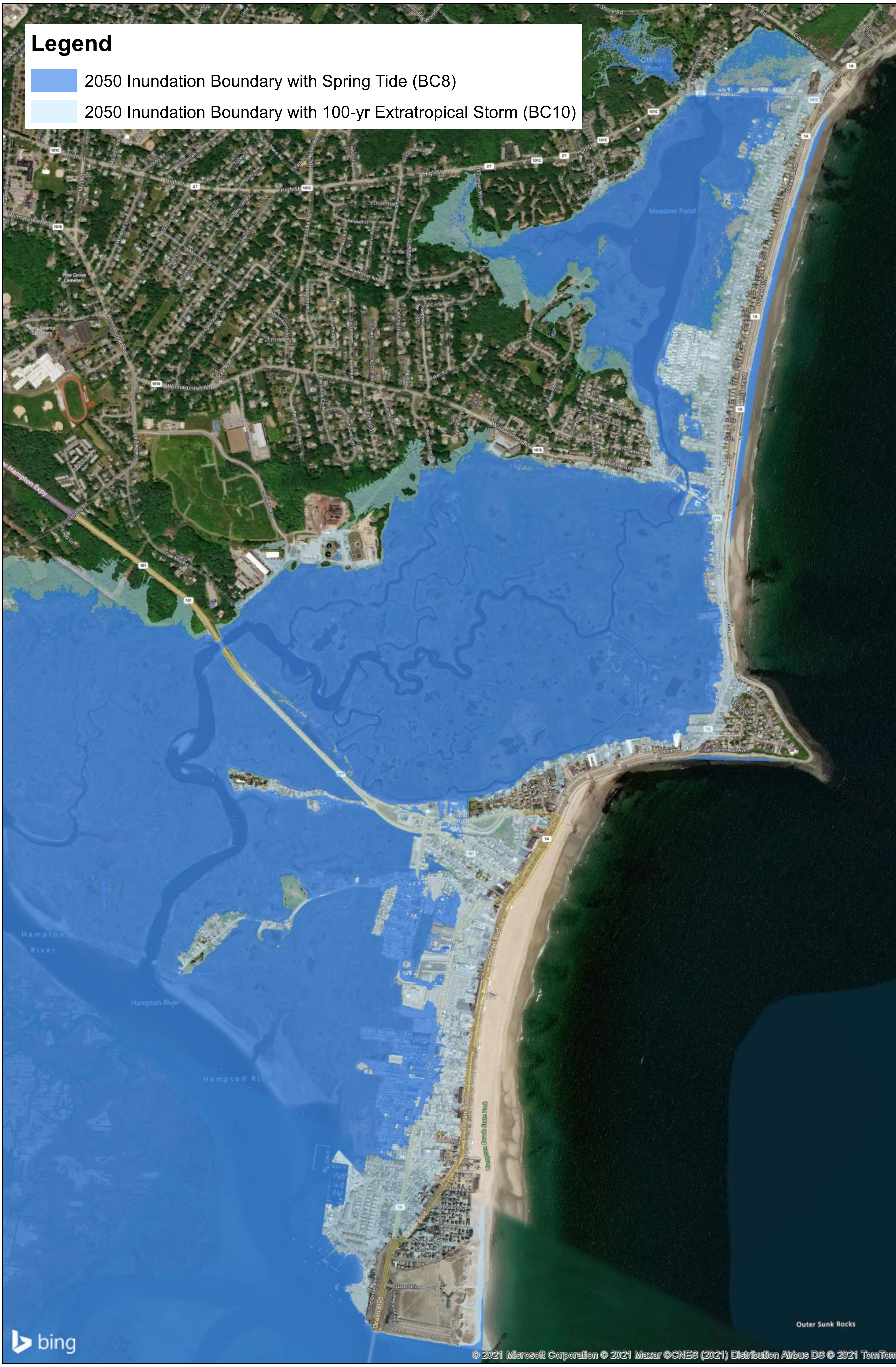
MILONE & MACBROOM
Now part of SLR
1 South Main St
Waterbury, VT 05676
802-882-8335



HYDRAULIC MODEL RESULTS - FUTURE TIDES

HAMPTON FLOOD STUDY
TOWN OF HAMPTON, NH
(MMI, 2021)

MILONE & MACBROOM
1 South Main St
Waterbury, VT 05676
802-882-8335



HYDRAULIC MODEL RESULTS - 2050 FUTURE TIDES WITH EXTRATROPICAL STORM

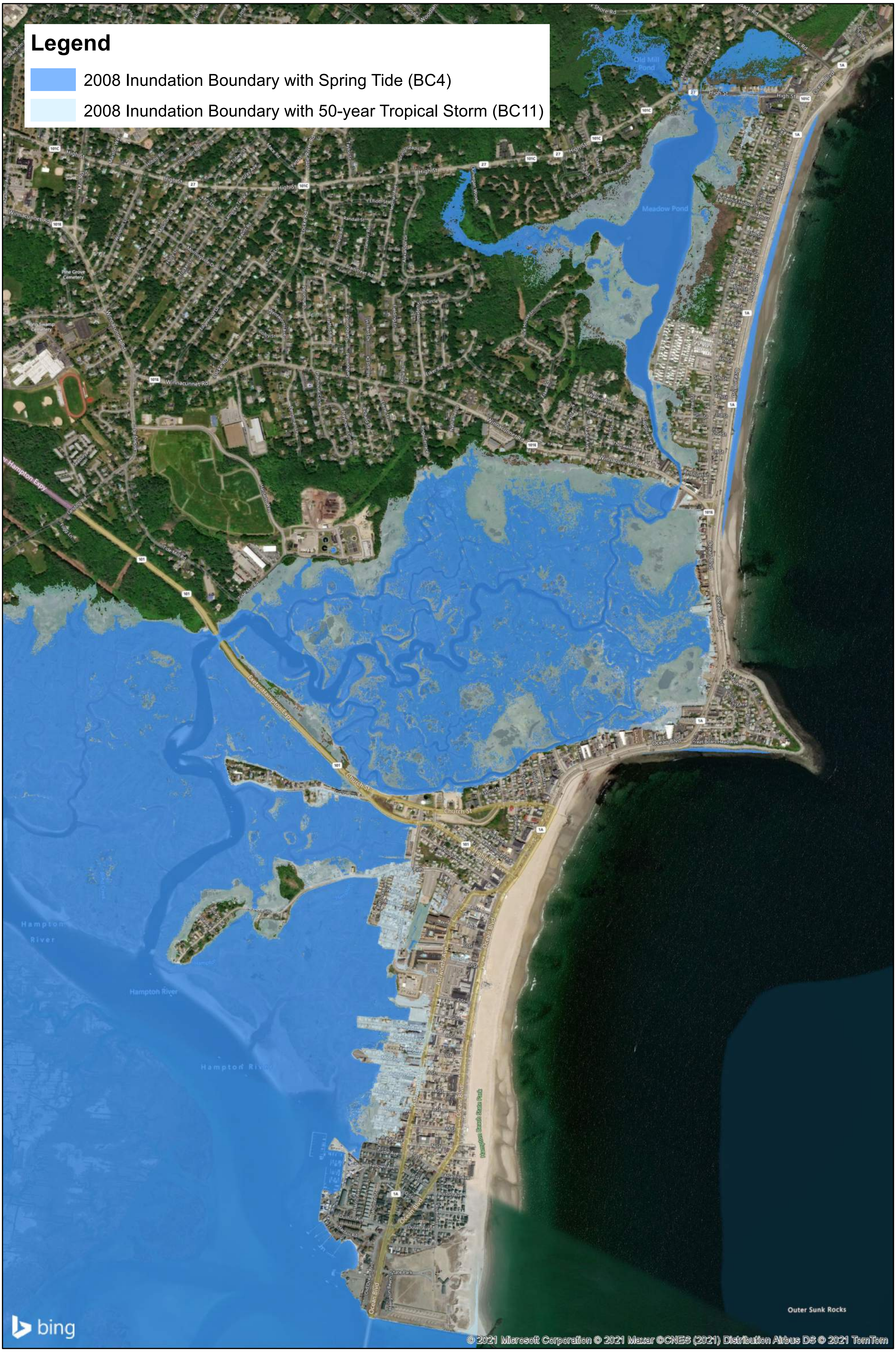
HAMPTON FLOOD STUDY
TOWN OF HAMPTON, NH
(MMI, 2021)

0 225 450 900 1,350 1,800
Feet

1 in = 1,125 feet

N

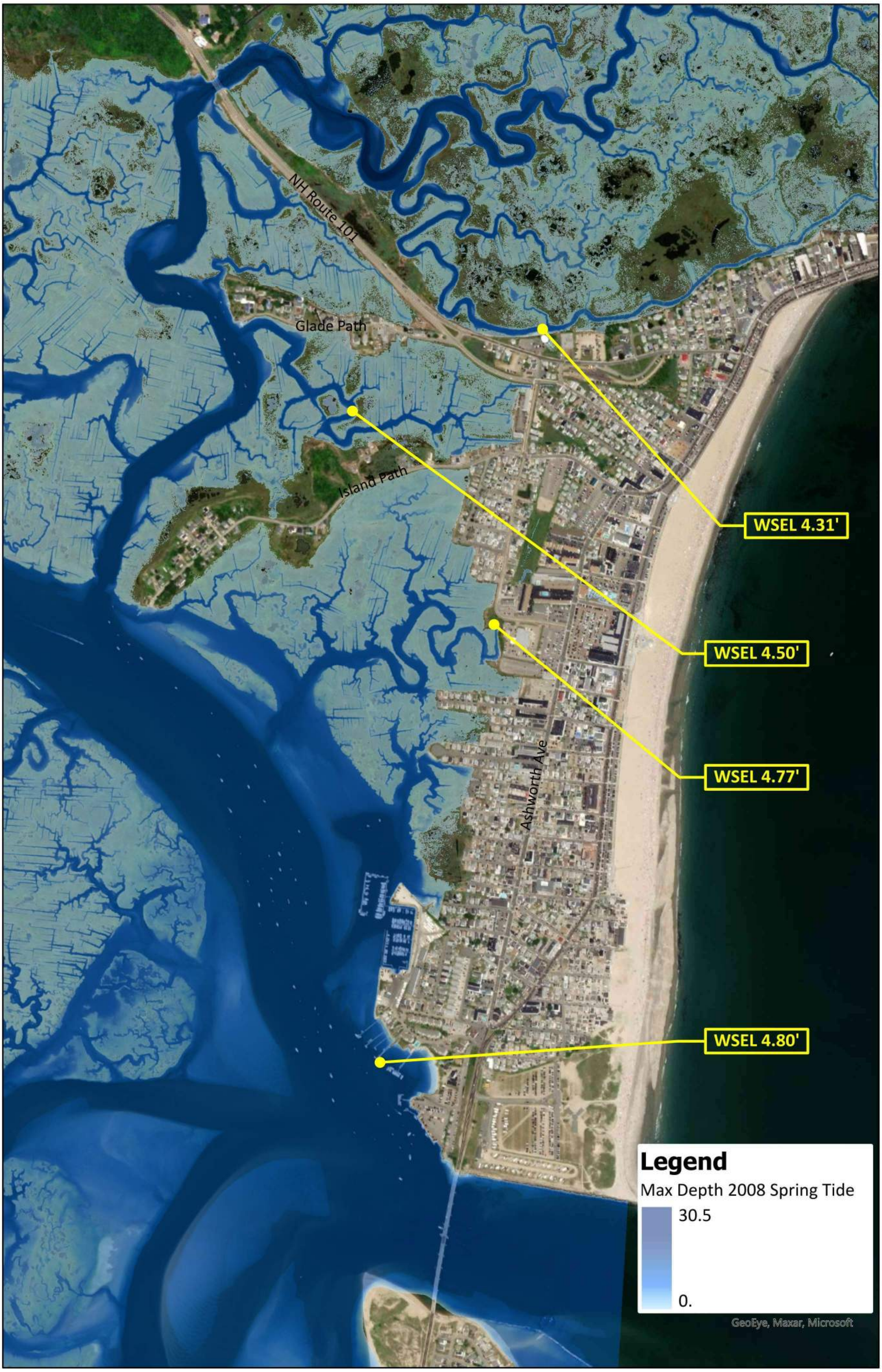
MILONE & MACBROOM
Now part of SLR
1 South Main St
Waterbury, VT 05676
802-882-8335



HYDRAULIC MODEL RESULTS - PRESENT TIDES WITH TROPICAL STORM

HAMPTON FLOOD STUDY
TOWN OF HAMPTON, NH
(MMI, 2021)

MILONE & MACBROOM — Now part of SLR
1 South Main St
Waterbury, VT 05676
802-882-8335



HYDRAULIC MODEL EXISTING RESULTS:
2008 SPRING TIDE

HAMPTON HARBOR FLOOD STUDY
TOWN OF HAMPTON, NH

0 400 800 1,200 Feet

1 in = 800 feet

N

Hoyle, Tanner & Associates, Inc.

HYDRAULIC MODEL EXISTING RESULTS:
2019 KING TIDE

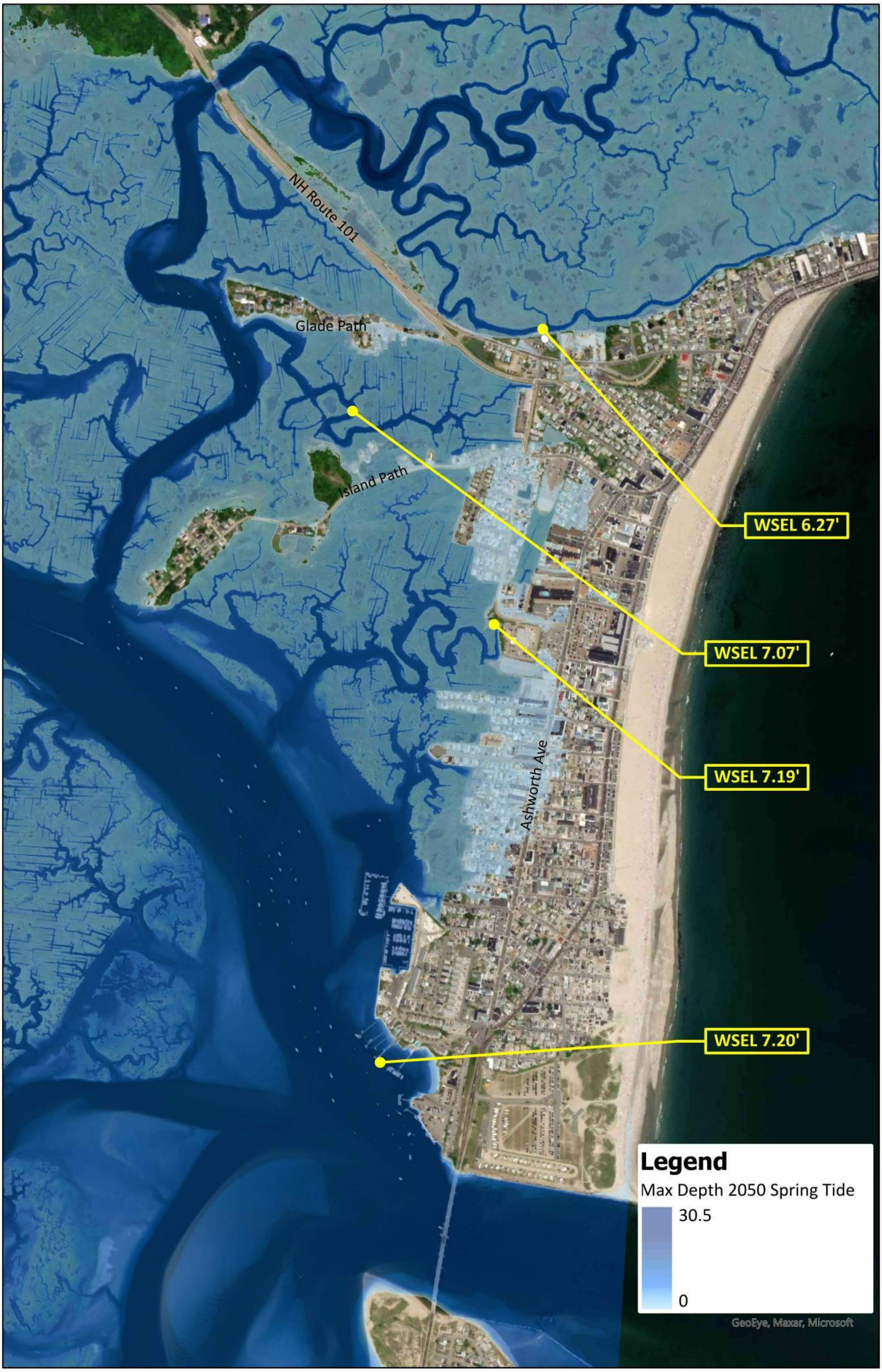
HAMPTON HARBOR FLOOD STUDY
TOWN OF HAMPTON, NH

0 400 800 1,200 Feet

1 in = 800 feet

N

Hoyle, Tanner & Associates, Inc.



HYDRAULIC MODEL EXISTING RESULTS:
2050 SPRING TIDE

HAMPTON HARBOR FLOOD STUDY
TOWN OF HAMPTON, NH

0 400 800 1,200 Feet

1 in = 800 feet

N

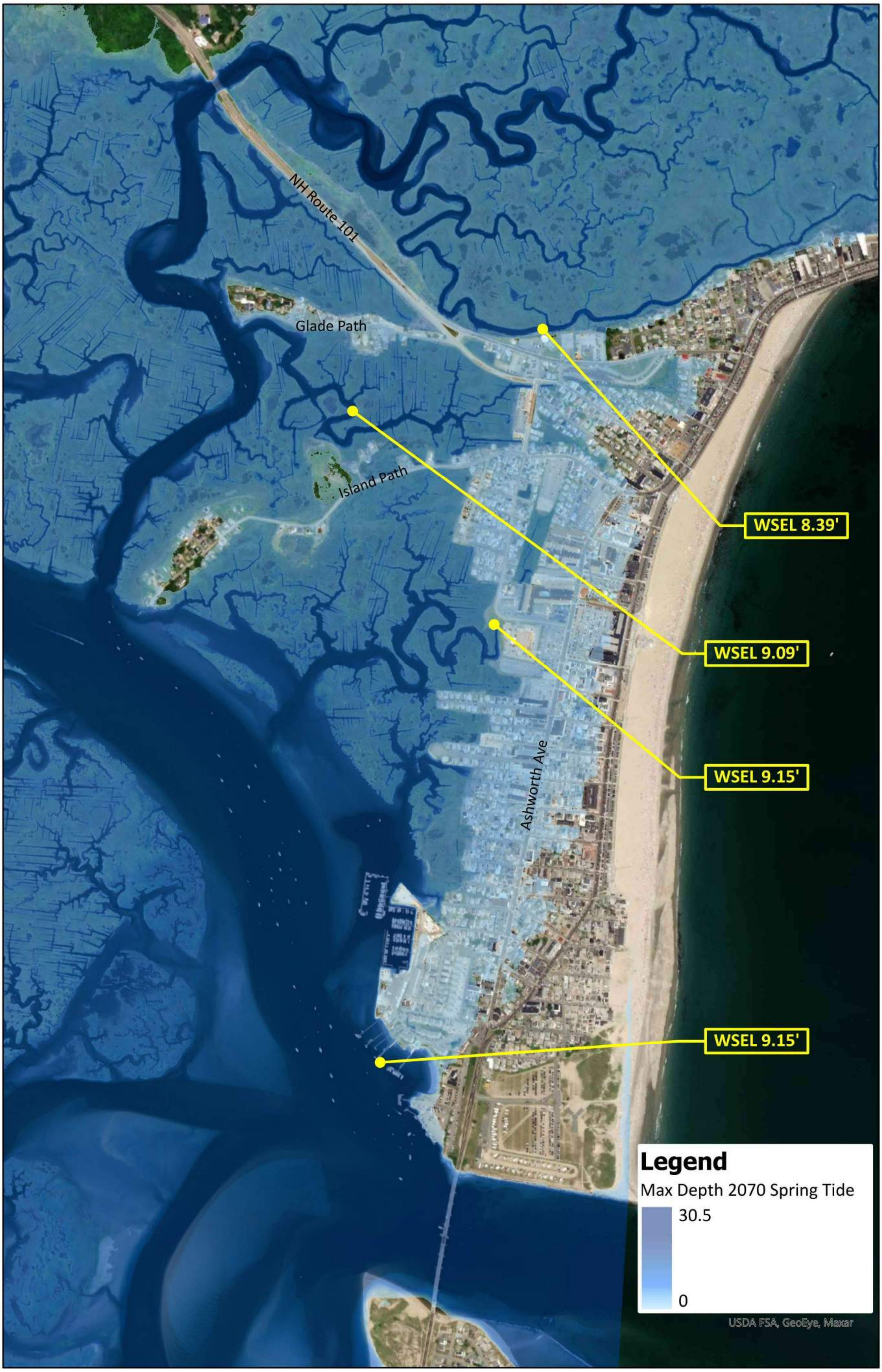
Hoyle, Tanner & Associates, Inc.

HYDRAULIC MODEL EXISTING RESULTS:
2050 EXTRATROPICAL STORM
HAMPTON HARBOR FLOOD STUDY
TOWN OF HAMPTON, NH

0 400 800 1,200 Feet
1 in = 800 feet

N

Hoyle, Tanner & Associates, Inc.



HYDRAULIC MODEL EXISTING RESULTS:
2070 SPRING TIDE

HAMPTON HARBOR FLOOD STUDY
TOWN OF HAMPTON, NH

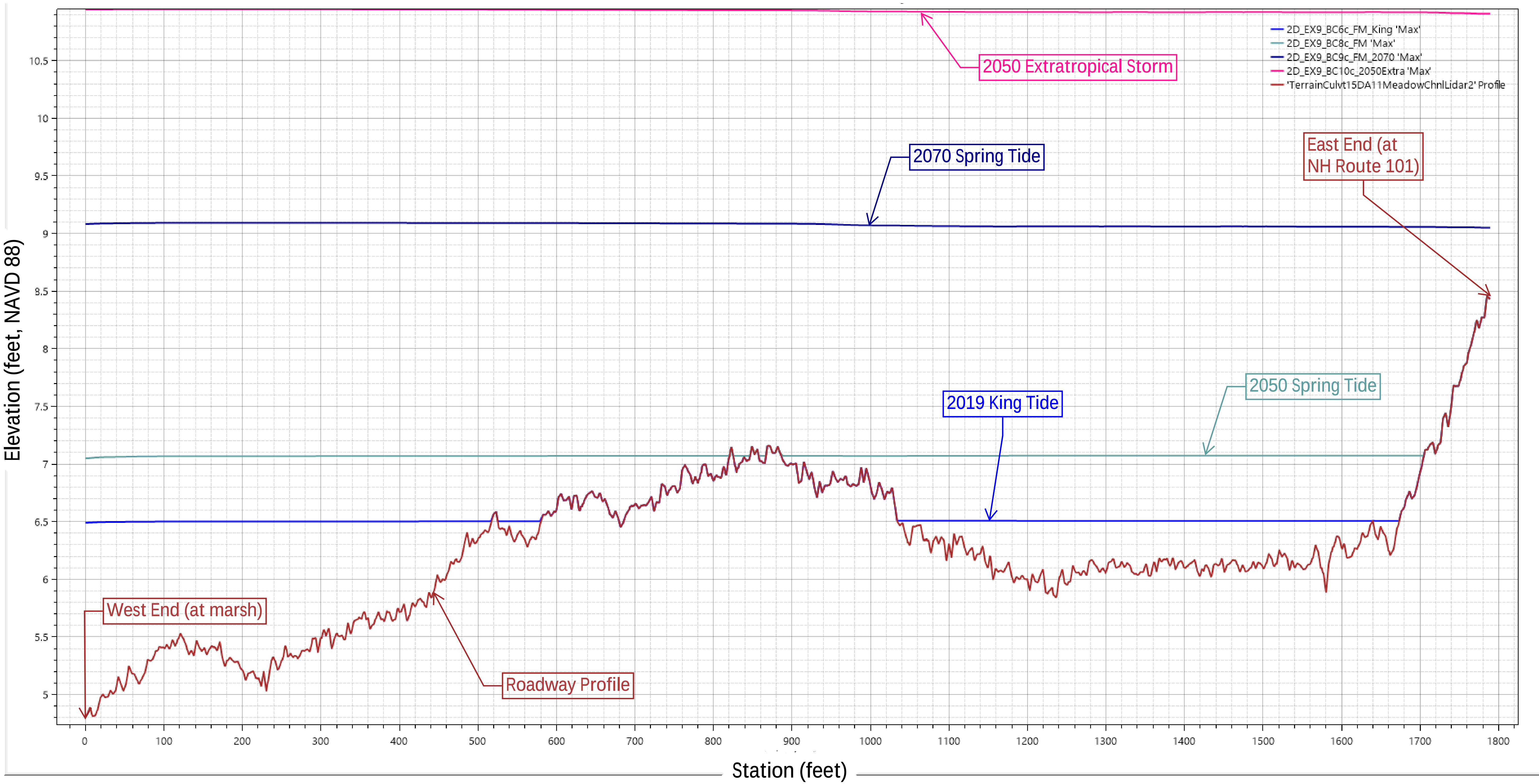
0 400 800 1,200 Feet

1 in = 800 feet

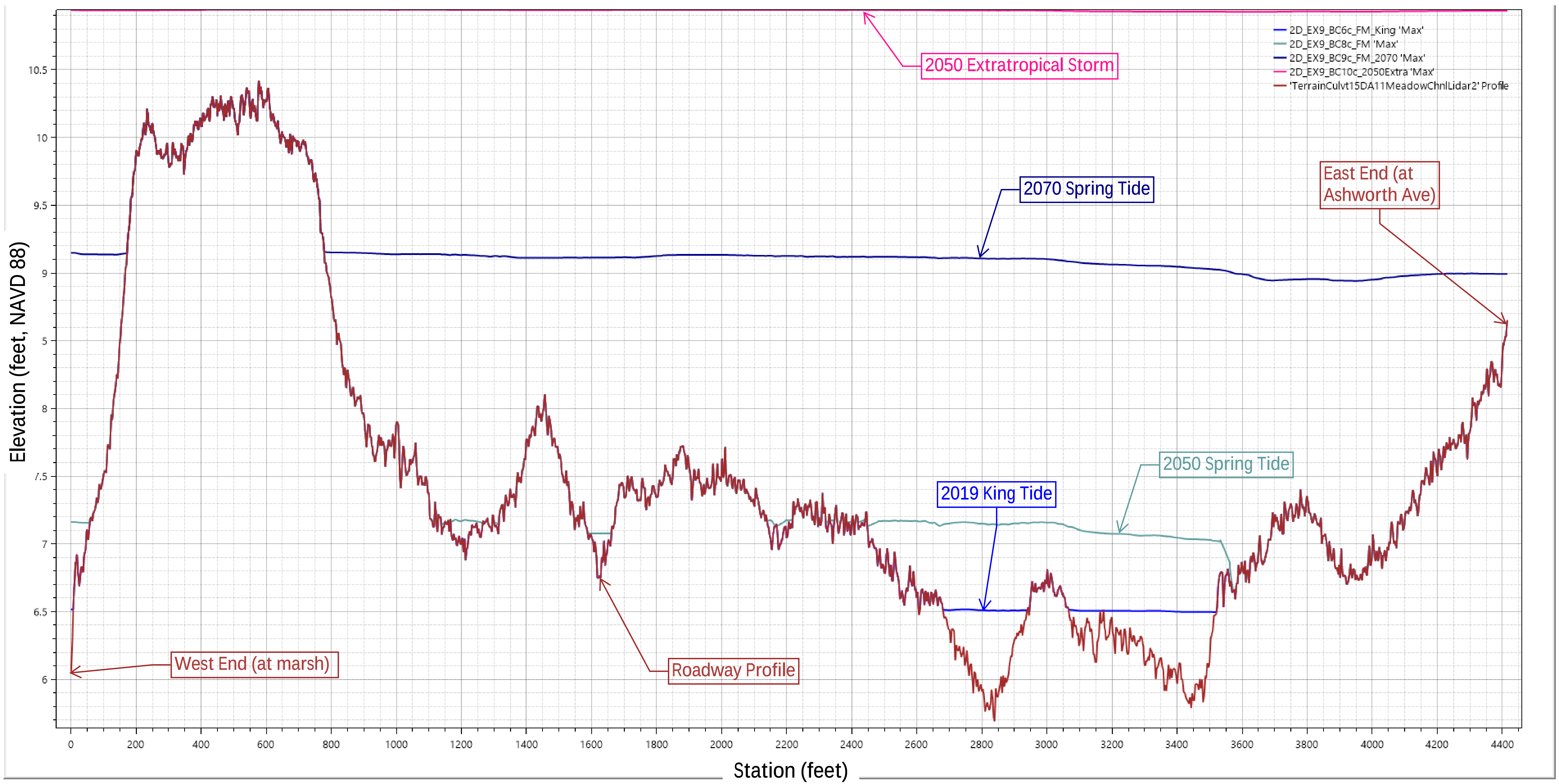
N

Hoyle, Tanner & Associates, Inc.

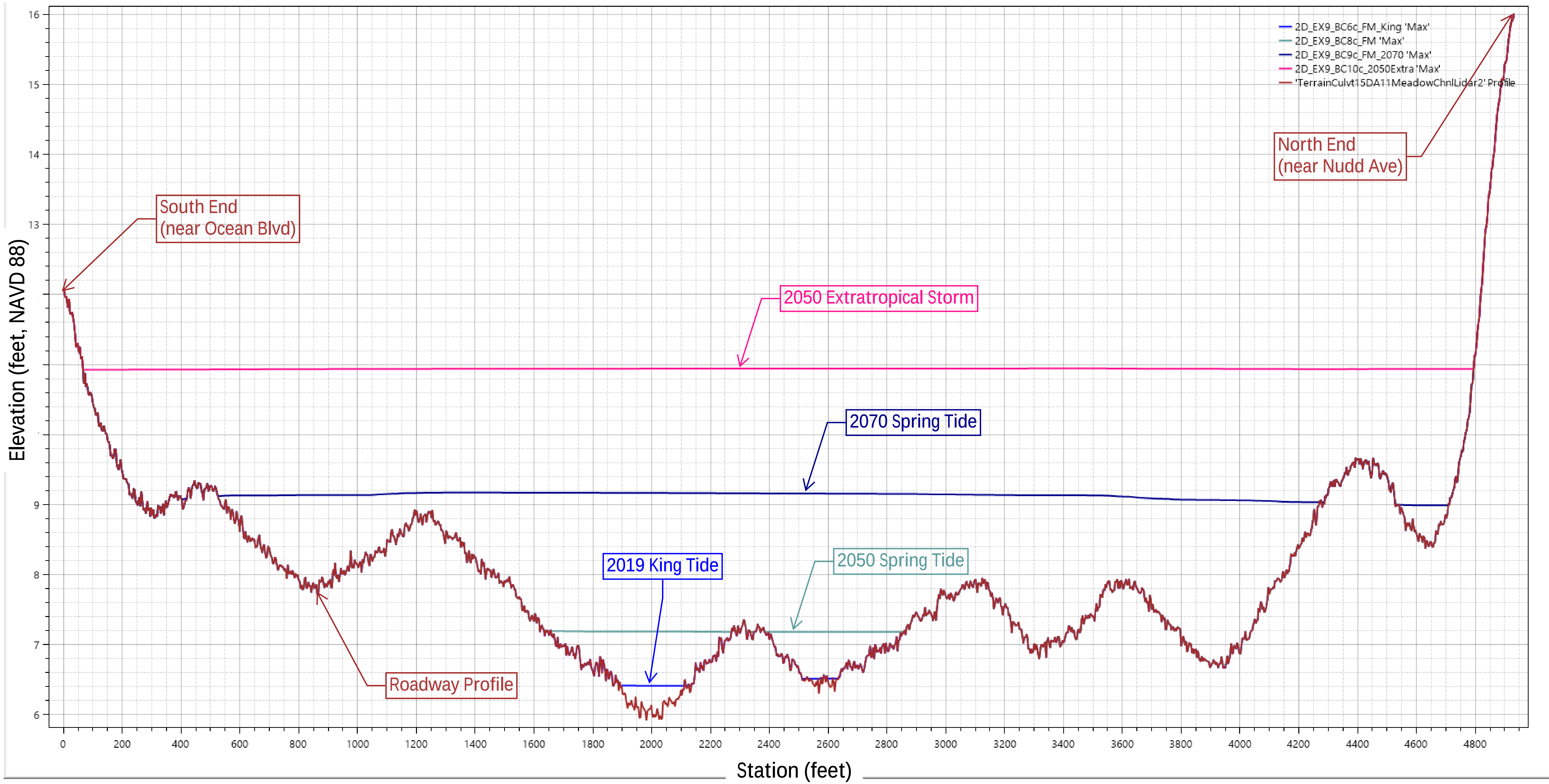
Glade Path Roadway Profile with Water Surface Elevations



Island Path Roadway Profile with Water Surface Elevations



Ashworth Ave Roadway Profile with Water Surface Elevations



APPENDIX F

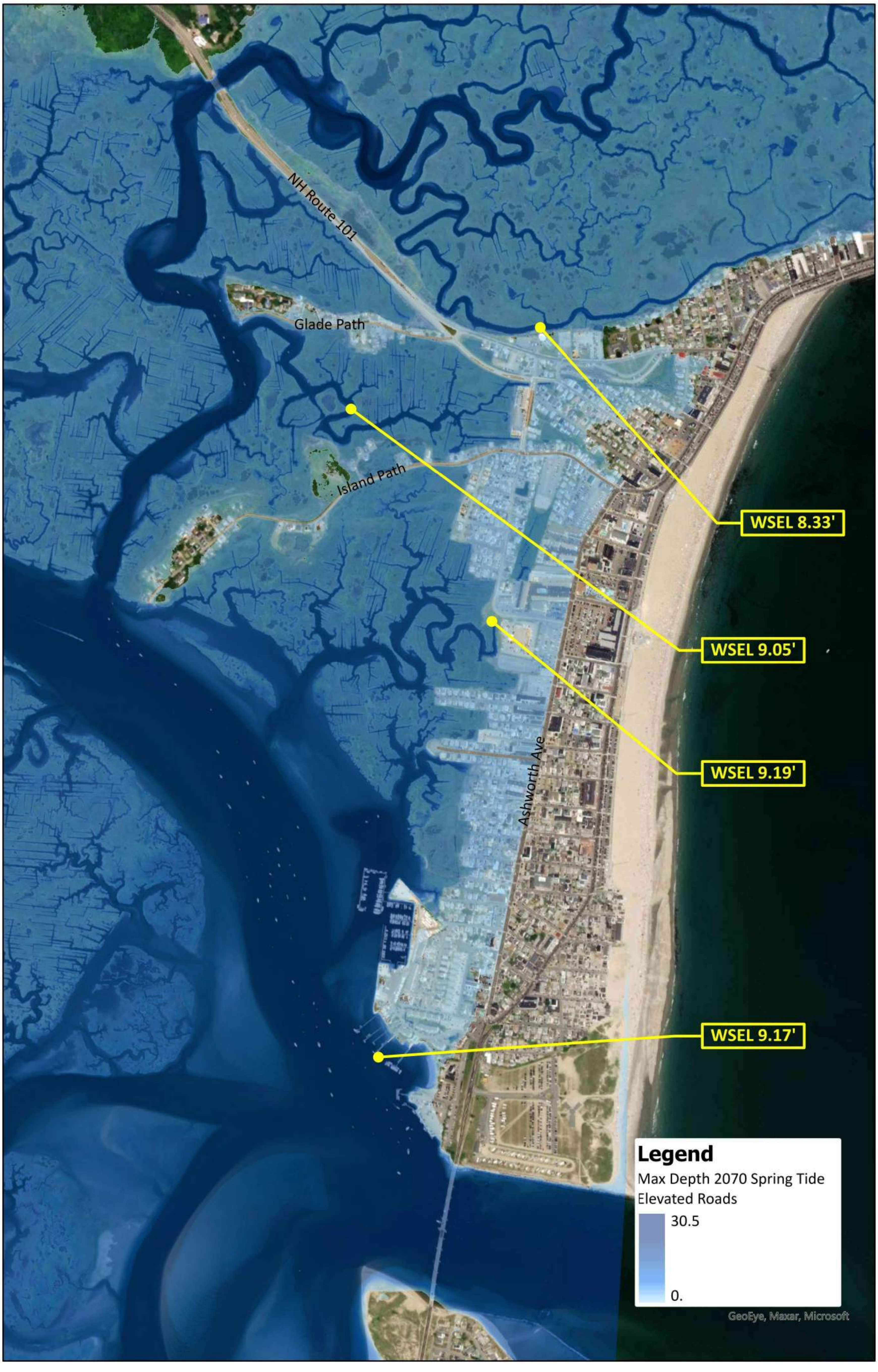
ALTERNATIVE CONDITIONS HYDRAULICS

ELEVATE FLOOD-PRONE ROADS AND PERMANENT BARRIERS

Engineering Report

March 2021

Hoyle, Tanner & Associates, Inc.



HYDRAULIC MODEL ALTERNATIVE RESULTS:
2070 SPRING TIDE ELEVATED ROADS

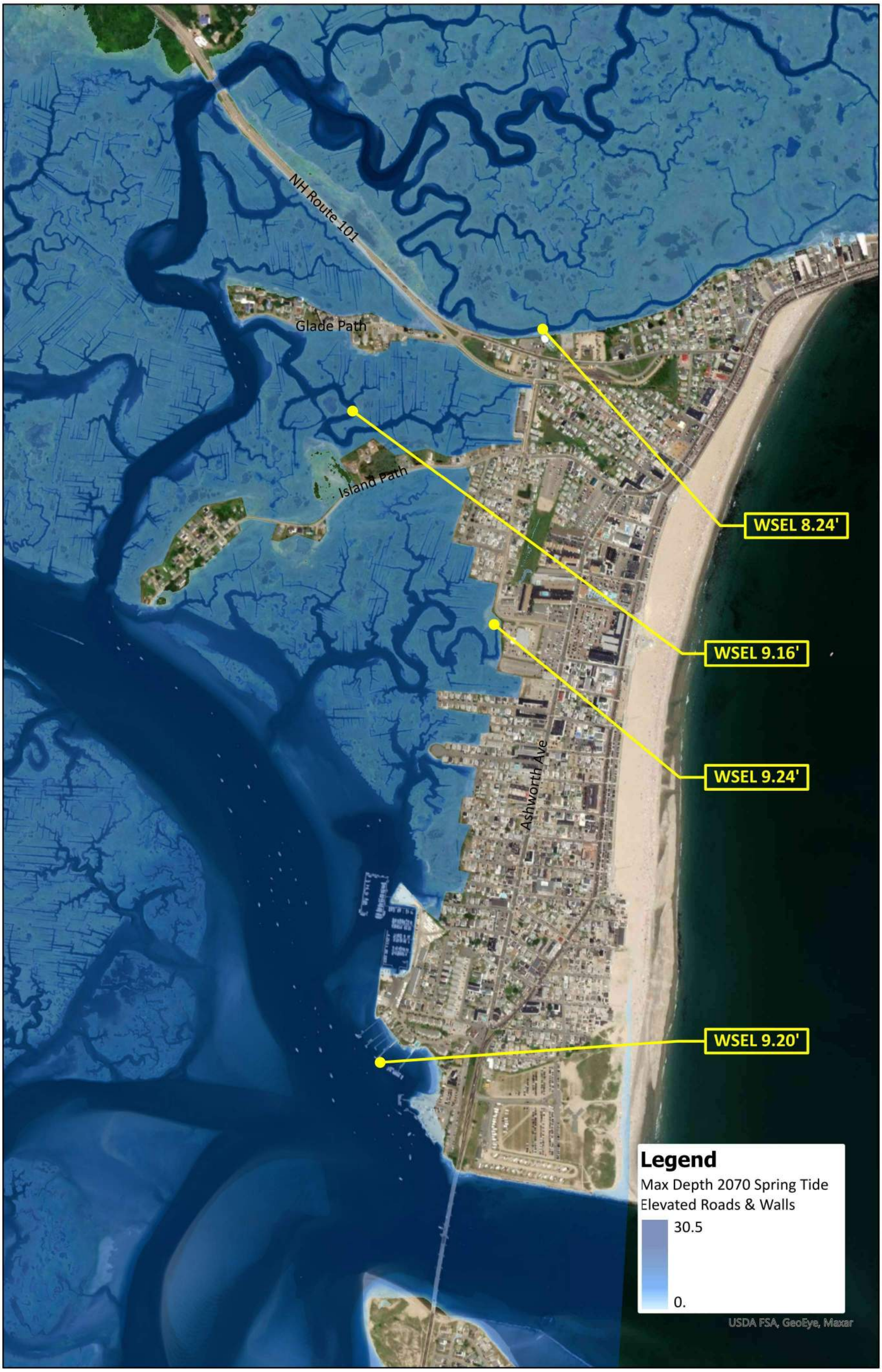
HAMPTON HARBOR FLOOD STUDY
TOWN OF HAMPTON, NH

0 400 800 1,200 Feet

1 in = 800 feet

N

Hoyle, Tanner & Associates, Inc.



HYDRAULIC MODEL ALTERNATIVE RESULTS:
2070 SPRING TIDE ELEVATED ROADS & WALLS
HAMPTON HARBOR FLOOD STUDY
TOWN OF HAMPTON, NH

0 400 800 1,200 Feet
1 in = 800 feet

N

Hoyle, Tanner & Associates, Inc.

APPENDIX G

FEMA FIS & FIRM

Engineering Report

March 2021

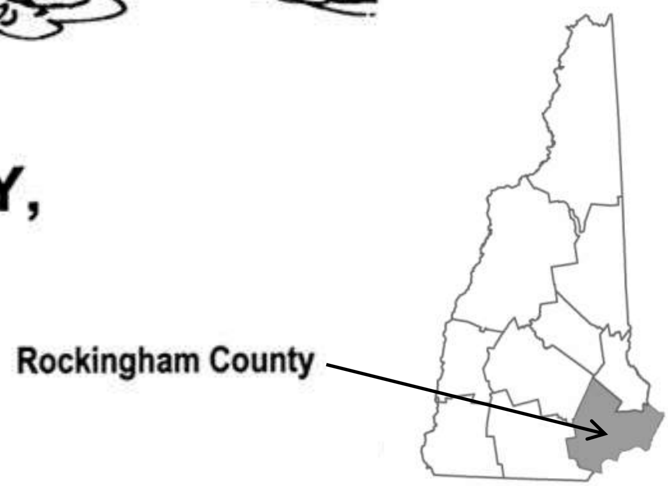
Hoyle, Tanner & Associates, Inc.

FLOOD INSURANCE STUDY

VOLUME 1 OF 3

ROCKINGHAM COUNTY, NEW HAMPSHIRE

(ALL JURISDICTIONS)



COMMUNITY NAME	COMMUNITY NUMBER	COMMUNITY NAME	COMMUNITY NUMBER
Atkinson, Town of	330175	New Castle, Town of	330135
Auburn, Town of	330176	Newfields, Town of	330228
Brentwood, Town of	330125	Newington, Town of	330229
Candia, Town of	330126	Newmarket, Town of	330136
Chester, Town of	330182	Newton, Town of	330240
Danville, Town of	330199	North Hampton, Town of	330232
Deerfield, Town of	330127	Northwood, Town of	330855
Derry, Town of	330128	Nottingham, Town of	330137
East Kingston, Town of	330203	Plaistow, Town of	330138
Epping, Town of	330129	Portsmouth, City of	330139
Exeter, Town of	330130	Raymond, Town of	330140
Fremont, Town of	330131	Rye, Town of	330141
Greenland, Town of	330210	Salem, Town of	330142
Hampstead, Town of	330211	Sandown, Town of	330191
Hampton Falls, Town of	330133	Seabrook Beach Village District	330854
Hampton, Town of	330132	Seabrook, Town of	330143
Kensington, Town of	330216	South Hampton, Town of	330193
Kingston, Town of	330217	Stratham, Town of	330197
Little Boar's Head, Village District of	330856	Windham,Town of	330144
Londonderry, Town of	330134		

REVISED
January 29, 2021

Federal Emergency Management Agency

FLOOD INSURANCE STUDY NUMBER
33015CV001B

NOTICE TO
FLOOD INSURANCE STUDY USERS

Communities participating in the National Flood Insurance Program have established repositories of flood hazard data for floodplain management and flood insurance purposes. This Flood Insurance Study (FIS) may not contain all data available within the repository. It is advisable to contact the community repository for any additional data.

Part or all of this FIS may be revised and republished at any time. In addition, part of this Preliminary FIS may be revised by the Letter of Map Revision process, which does not involve republication or redistribution of the FIS. It is, therefore, the responsibility of the user to consult with community officials and to check the community repository to obtain the most current FIS components.

Initial Countywide FIS Effective Date: May 17, 2005

Revised Countywide FIS Effective Date: January 29, 2021

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	Bryant Brook	Profiles	21P-22P
	Cohas Brook	Profiles	23P-24P
	Cunningham Brook	Profiles	25P-34P
	Drew Brook	Profiles	35P-37P
	Dudley Brook	Profiles	38P-41P
	Exeter River	Profiles	42P-51P
	Flatrock Brook	Profiles	52P-56P
	Golden Brook	Profiles	57P-63P
	Grassy Brook	Profile	64P
	Hidden Valley Brook	Profiles	65P-68P
	Hill Brook	Profile	69P
	Hog Hill Brook	Profiles	70P-71P
	Hornes Brook	Profiles	72P-75P
	Kelly Brook	Profiles	76P-77P
	Lamprey River (Town of Newmarket)	Profiles	78P
	Lamprey River	Profiles	79P-88P
	Little Cohas Brook	Profiles	89P-99P
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	Upper Beaver Brook	Profiles	196P-198P
	Wash Pond Tributary	Profile	199P
	West Channel Policy Brook	Profiles	200P-201P
	Winnicut River	Profile	202P
Exhibit 2 -	Flood Insurance Rate Map Index		
	Flood Insurance Rate Map		

FLOOD INSURANCE STUDY
ROCKINGHAM COUNTY, NEW HAMPSHIRE
(ALL JURISDICTIONS)

1.0 INTRODUCTION

1.1 Purpose of Study

This countywide Flood Insurance Study (FIS) investigates the existence and severity of flood hazards in, or revises and updates previous FISs/Flood Insurance Rate Maps (FIRMs) for, the geographic area of Rockingham County, including: the City of Portsmouth; the Towns of Atkinson, Auburn, Brentwood, Candia, Chester, Danville, Deerfield, Derry, East Kingston, Epping, Exeter, Fremont, Greenland, Hampstead, Hampton, Hampton Falls, Kensington, Kingston, Londonderry, New Castle, Newfields, Newington, Newmarket, Newton, North Hampton, Northwood, Nottingham, Plaistow, Raymond, Rye, Sandown, Salem, Seabrook, South Hampton, Stratham, and Windham; and the Seabrook Beach Village District (hereinafter referred to collectively as Rockingham County).

This FIS aids in the administration of the National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973. This study has developed flood risk data for various areas of the county that will be used to establish actuarial flood insurance rates. This information will also be used by the communities of Rockingham County to update existing floodplain regulations as part of the Regular Phase of the National Flood Insurance Program (NFIP), and by local and regional planners to further promote sound land use and floodplain development. Minimum floodplain management requirements for participation in the NFIP are set forth in the Code of Federal Regulations at 44 CFR, 60.3.

In some States or communities, floodplain management criteria or regulations may exist that are more restrictive or comprehensive than the minimum Federal requirements. In such cases, the more restrictive criteria take precedence and the State (or other jurisdictional agency) will be able to explain them.

This FIS report presents the contents of original community-based FIS reports as well as two updates. The first update was completed in 2005, when the community reports were combined into a countywide report and the Flood Insurance Rate Maps were presented in digital format. The second update was completed in 2013, when new coastal and riverine analyses were performed in 14 coastal communities in the eastern portion of Rockingham County.

Additional information regarding the 2013 update is included under the heading "The January 29, 2021 Countywide Revision" located within appropriate sections throughout this report.

1.2 Authority and Acknowledgments

The sources of authority for this FIS are the National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973.

The community based FIS reports prior to 1979 were prepared for the Federal Insurance Administration (FIA). In 1979, an executive order merged the FIA into the newly formed Federal Emergency Management Agency (FEMA). Reports from that date forward were prepared for FEMA.

The May 17, 2005 FIS (FEMA, 2005) was prepared to include the incorporated communities within Rockingham County in a countywide FIS. Information on the authority and acknowledgments for each jurisdiction included in the 2005 countywide FIS, as compiled from their previously printed FIS reports, is shown below.

Atkinson, Town of:	The hydrologic and hydraulic analyses for the FIS report dated April 2, 1993, were prepared by the U.S. Geological Survey (USGS) for the Federal Emergency Management agency (FEMA), under Inter-Agency Agreement No. EMW-88-E-2738, Project Order No. 4. That work was completed in August 1991. The hydrologic and hydraulic analyses for Island Pond were taken from the FIS for the Town of Derry (FEMA, 1981). The hydrologic and hydraulic analyses for Bryant Brook were taken from the FIS for the Town of Plaistow (FEMA, April1981).
Brentwood, Town of:	The hydrologic and hydraulic analyses for the FIS report dated October 15, 1980, were prepared by the Soil Conservation Service (SCS) for the Federal Insurance Administration (FIA), under Inter-Agency Agreement No. IAA-H-17-78. That work was completed in May 1979. The hydrologic and hydraulic analyses for the FIS report dated May 4, 2000, were prepared by the USGS for FEMA, under Inter-Agency Agreement No. EMW-97-1A-0155, Project Order No. 1. That work was completed in June 1998.
Derry, Town of:	The hydrologic and hydraulic analyses for the FIS report dated April 15, 1980, were prepared by Anderson-Nichols and Company, Inc., for the FIA, under Contract No. H-3989. That work was completed in March 1978.
Epping, Town of:	The hydrologic and hydraulic analyses for the FIS report dated October 15, 1981, were performed by the SCS for FEMA, under Inter-Agency Agreement No. IAA-H-17-78, Project Order No. 15. That work was completed in September 1979.

Exeter, Town of:	The hydrologic and hydraulic analyses for the FIS report dated November 17, 1981, were prepared by Stone & Webster Engineering Corporation for FEMA, under Contract No. H-4772. That work was completed in May 1980.
Fremont, Town of:	The hydrologic and hydraulic analyses for the FIS report dated June 19, 1989, represent a revision of the original analyses prepared by the SCS for FEMA, under Inter-Agency Agreement No. IAA-H-17-78, Project Order No. 15. The work for the original analyses was completed in May 1979. The hydrologic and hydraulic analyses for Spruce Swamp were prepared by Dewberry & Davis LLC, under agreement with FEMA. That work was completed in June 1988.
Greenland, Town of:	The hydrologic and hydraulic analyses for the FIS report dated May 17, 1989, were performed by the SCS for FEMA, under Inter-Agency Agreement No. EMW-86-E-2225, Project Order No. 01. That work was completed in September 1987
Hampstead, Town of:	The hydrologic and hydraulic analyses for the FIS report dated June 16, 1993, were prepared by the USGS for FEMA, under Inter-Agency Agreement No. EMW-88-E-2738, Project Order No. 4. That work was completed in August 1991. The flooding information for Island Pond was taken from the FIS for the Town of Derry (FEMA, 1981).
Hampton, Town of:	The hydrologic and hydraulic analyses for the FIS report dated July 3, 1986, were prepared by Stone & Webster Engineering Corporation for FEMA, under Contract No. H-4772. That work was completed in January 1984.
Hampton Falls, Town of:	The hydrologic and hydraulic analyses for the FIS report dated October 15, 1981, were prepared by Stone & Webster Engineering Corporation for FEMA, under Contract No. H-4772. That work was completed in April 1980.
Kingston, Town of:	The hydrologic and hydraulic analyses for the FIS report dated April 15, 1992, were prepared by the USGS for FEMA, under Inter-Agency Agreement No. EMW-87-E-2548, Project Order No. 1A. That work was completed in July 1989.

Londonderry, Town of:	The hydrologic and hydraulic analyses for the FIS report dated May 5, 1980, were prepared by Anderson-Nichols & Company, Inc., for the FIA, under Contract No. H-3989. That work was completed in March 1978.
New Castle, Town of:	The hydrologic and hydraulic analyses for the FIS report dated August 5, 1986, were prepared by Stone & Webster Engineering Corporation for FEMA, under Contract No. H-4772. That work was completed in April 1984.
Newfields, Town of:	The hydrologic and hydraulic analyses for the FIS report dated June 5, 1989, were prepared by the SCS for FEMA, under Inter-Agency Agreement No. EMW-86-E-2225, Project Order No. 01. That work was completed in September 1987.
Newmarket, Town of:	The hydrologic and hydraulic analyses for the FIS report dated May 2, 1991, were prepared by the USGS for FEMA, under Inter-Agency Agreement No. EMW-85-E-1823, Project Order No. 20. That work was completed in August 1989.
North Hampton, Town of:	The hydrologic and hydraulic analyses for the FIS report dated June 3, 1986, were prepared by Stone & Webster Engineering Corporation for FEMA, under Contract No. H-4772. That work was completed in February 1984. The Village District of Little Boar's Head, which was enrolled on June 27, 2017, is located on the coastal portion of what was formerly a portion of North Hampton.
Plaistow, Town of:	The hydrologic and hydraulic analyses for the FIS report dated October 15, 1980, were prepared by Anderson-Nichols & Company, Inc., for the FIA, under Contract No. H-4589. Approximate flood boundaries for portions of Seaver Brook and several unnamed streams and swampy areas were determined in August 1976, by Michael Baker, Jr. Inc., under contract to the FIA. That work was completed in October 1978.
Portsmouth, City of:	The hydrologic and hydraulic analyses for the FIS report dated November 17, 1981, were prepared by Stone & Webster Engineering Corporation for FEMA, under Contract No. H-4772. That work was completed in April 1980.

Raymond, Town of:	The hydrologic and hydraulic analyses for the FIS report dated October 15, 1981, were prepared by the SCS for FEMA, under Inter-Agency Agreement No. IAA-H-17-78. That work was completed in September 1979. The hydrologic and hydraulic analyses for the FIS report dated April 15, 1992, were prepared by Rivers Engineering Corporation for FEMA, under Contract No. EMW-89-C-2821, Project Order No. R89508. That work was completed October 1989. The hydrologic and hydraulic analyses for the FIS report dated May 2, 1995, were prepared by Roald Haestad, Inc., for FEMA, under Contract No. EMW-90-C-3126. That work was completed in March 1993.
Rye, Town of:	The hydrologic and hydraulic analyses for the FIS report dated June 17, 1986, were prepared by Stone & Webster Engineering Corporation for FEMA, under Contract No. H-4772. That work was completed in March 1984.
Salem, Town of:	The hydrologic and hydraulic analyses for the December 1978 FIS report and June 15, 1979, FIRM (hereinafter referred to as the 1979 FIS), were prepared by the U. S. Army Corps of Engineers (USACE), New England District, for the FIA, under Inter-Agency Agreement No. 1AA-H-7-76, Project Order No. 24. That work was completed in August 1977. The hydrologic and hydraulic analyses for the FIS report dated April 6, 1998 were prepared by the U. S. Department of Agriculture, Natural Resources Conservation Service (NRCS), for FEMA, under Contract No. EMW-94-E-4437. That work was completed in September 1995.
Seabrook, Town of:	The hydrologic and hydraulic analyses for the FIS report dated June 17, 1986, were prepared by Stone & Webster Engineering Corporation for FEMA, under Contract No. H-4772. That work was completed in December 1983.
Seabrook Beach Village District:	The hydrologic and hydraulic analyses for the FIS report dated August 5, 1986, were performed during the preparation of the FIS for the Town of Seabrook by Stone & Webster Engineering Corporation for FEMA, under Contract No. H-4772. The Town of Seabrook study was completed in December 1983.

South Hampton, Town of:	The hydrologic and hydraulic analyses for the FIS report dated July 15, 1992, were prepared by the USGS for FEMA, under Inter-Agency Agreement No. EMW-89-E-2997, Project Order No. 5. That work was completed in September 1990.
Stratham, Town of:	The hydrologic and hydraulic analyses for the FIS report dated May 17, 1989, were prepared by the SCS for FEMA, under Inter-Agency Agreement No. EMW-86-E-2225, Project Order No. 1. That work was completed in September 1987.
Windham, Town of:	The hydrologic and hydraulic analyses for the FIS report dated were performed by Anderson-Nichols & Company, Inc., for the FIA, under Contract No. H-3989. That work was completed in March 1978.

The authority and acknowledgments for the Towns of Auburn, Candia, Chester, Danville, Deerfield, East Kingston, Kensington, Newington, Northwood, Nottingham, and Sandown were not available prior to the 2005 countywide study because no FIS reports had been published for those communities.

The 2005 countywide FIS was produced by Dewberry & Davis LLC under agreement with FEMA. The work was effective in May of 2005. The contract required the digital conversion of existing effective FIRMs and Flood Hazard Boundary Maps, and the preparation of a FIS and Digital FIRM (DFIRM) for Rockingham County (All Jurisdictions). No new hydrologic or hydraulic analyses were prepared.

Base map information shown on FIRM panels produced for the 2005 study was derived from USGS Digital Orthophoto Quadrangles (DOQs) produced at a scale of 1:12,000 from photography dated 1998 or later.

The digital FIRM was produced using New Hampshire State Plane Coordinate system, FIPS Zone 2800 Feet, referenced to the North American Datum of 1983 (NAD 83), GRS80 spheroid.

The January 29, 2021 Countywide Revision

The January 29, 2021 countywide revision was prepared by the University of New Hampshire (UNH) for FEMA under Agreement No. EMB-2010-CA-0916 and completed in November of 2019. The study consisted of revisions to the coastal and riverine analyses in 14 contiguous communities located in eastern Rockingham County, including the City of Portsmouth and the Towns of Exeter, Greenland, Hampton, Hampton Falls, New Castle, Newfields, Newington, Newmarket, North Hampton, Rye, Seabrook, Seabrook Beach Village District, and Stratham.

The January 29, 2021 countywide revision FIS includes revisions to detailed riverine studies in the incorporated community of Newmarket, NH within Rockingham County. Information on the authority and acknowledgements for each of these jurisdictions included in this FIS is shown below.

Newmarket, Town of:	The hydrologic and hydraulic analyses for the FIS report dated January 29, 2021, were prepared by the U.S. Geological Survey, New England Water Science Center, for FEMA. That work was completed in November, 2012.

In addition, the January 29, 2021 countywide revision FIS includes revisions to Zone A study streams on updated panels in the City of Portsmouth and the Towns of Exeter, Greenland, Hampton, Hampton Falls, New Castle, Newfields, Newington, Newmarket, North Hampton, Rye, Seabrook, Stratham, and the Village Districts of Seabrook Beach and Little Boar's Head. The revisions were based on new estimates for the 1% flood discharges and delineating the 1% flood limits on better topography than available at the time of the previous studies. The work was completed in June 2013.

Base map information shown on FIRM panels produced for mainland NH for the 2013 revision was derived from 1-foot resolution orthophotography acquired in April-May, 2010. Base map information shown on FIRM panels produced for the Isles of Shoals in the Town of Rye was derived from 1-meter resolution orthophotography acquired in 2012. The projection used in the preparation of the digital FIRM was New Hampshire State Plane Feet, FIPS Zone 2800, referenced to the North American Datum of 1983 (NAD 83), GRS80 spheroid.

1.3 Coordination

During the early years of the National Flood Insurance Program, Consultation Coordination Officer's (CCO) meetings were held for each jurisdiction in this countywide FIS. An initial CCO meeting was held typically with representatives of FEMA, the community, and the study contractor to explain the nature and purpose of an FIS, and to identify the streams to be studied by detailed methods. A final CCO meeting was held typically with representatives of FEMA, the community, and the study contractor to review the results of the study.
Prior to the countywide FIS, the dates of the historical initial and final CCO

meetings held for all jurisdictions within Rockingham County are shown in Table 1, "Initial and Final CCO Meetings."

TABLE 1 – INITIAL AND FINAL CCO MEETINGS

Community Name	Initial CCO Meeting	Final CCO Meeting
Town of Atkinson	August 31, 1991	March 23, 1992
Town of Brentwood	July 15, 1997	*
Town of Derry	March 1976	February 13, 1979
Town of Epping	January 4, 1978	August 19, 1980
Town of Exeter	April 19, 1978	June 11, 1981
Town of Fremont	January 4, 1978	October 31, 1979
Town of Greenland	October 1, 1985	March 21, 1988
Town of Hampstead	August 31, 1987	January 21, 1992
Town of Hampton	April 19, 1978	January 16, 1985
Town of Hampton Falls	April 18, 1978	April 15, 1981
Town of Kingston	*	August 15, 1990
Village District of Little Boar's Head	April 19, 1978[1]	January 16, 1985[1]
Town of Londonderry	March 1976	March 28, 1979
Town of New Castle	April 19, 1978	January 21, 1985
Town of Newfields	October 22, 1985	July 8, 1988
Town of Newmarket	February 1985	April 4, 1990
Town of North Hampton	April 19, 1978	January 16, 1985
Town of Plaistow	*	September 10, 1979
City of Portsmouth	April 19, 1978	June 11, 1981
Town of Raymond	December 9, 1992	*
Town of Rye	April 19, 1978	April 12, 1985
Town of Salem	August 3, 1993	October 17, 1996
Town of Seabrook	April 18, 1978	December 5, 1984
Seabrook Beach Village District	*	September 11, 1985
Town of South Hampton	*	May 28, 1991
Town of Stratham	October 22, 1985	June 20, 1988
Town of Windham	March 1976	October 16, 1978

**Data not available*

[1] The land area for this community was previously shown as a portion of The Town of North Hampton. It has now been identified as a separate NFIP community. Therefore, the dates for this community were taken from the The Town of North Hampton.

For the 2005 countywide study, letters were sent to all communities within Rockingham County notifying them of the scope of the FIS. Letters were mailed on July 10, 2002, and stated that the effective FIRMs and Flood Hazard Boundary Maps (FHBMs) of these communities would be digitally converted to a format that conforms to FEMA's Digital FIRM (DFIRM) specifications. The letters further stated that no new hydrologic and hydraulic analyses were prepared. The results of the 2005 countywide study were reviewed at the final CCO meetings held on November 13, 2003, and attended by representatives of the communities, FEMA, Dewberry and Davis LLC, the University of New Hampshire, and the NH Office of State Planning.

For the January 29, 2021 countywide revision, invitations to attend a Risk MAP Discovery Meeting were sent to the 14 subject communities within Rockingham County on August 31, 2011. The invitations included a request to submit pertinent information on local flood risks and hazards to UNH. The meetings were held on September 22, 2011, and were attended by representatives of the communities, UNH, the FEMA Regional Service Center (RSC), FEMA, AECOM, the NH Office of State Planning, and the New Hampshire-Vermont Water Science Center of the U.S. Geological Survey. Prior to the release of the preliminary maps, communities were invited to attend one of a daylong series of Workmap review sessions held on August 1, 2013, and attended by representatives of the communities, the University of New Hampshire, FEMA, AECOM, the NH Office of Energy and Planning (formerly known as the NH Office of State Planning), and the New Hampshire-Vermont Water Science Center of the U.S. Geological Survey. The final CCO meetings were held on May 8, 2014, and attended by representatives of the communities, UNH, FEMA, AECOM, the NH Office of Energy and Planning, and USGS. All problems raised at that meeting were addressed in this study.

2.0 AREA STUDIED

2.1 Scope of Study

This FIS report covers the geographic area of Rockingham County, New Hampshire.

May 17, 2005 Countywide FIS

All or portions of the flooding sources listed in Table 2, "Flooding Sources Studied by Detailed Methods," were studied by detailed methods.

TABLE 2 – FLOODING SOURCES STUDIED BY DETAILED METHODS

Adams Pond	Lamprey River	Squamscott River
Atlantic Ocean	Little Cohas Brook	Taylor Brook (including Ballard Pond)
Beaver Brook	Little River No. 1	Taylor River
Beaver Lake	Little River No. 2	Tide Mill Creek
Black Brook	Little River No. 3	Tributary C to Beaver Brook
Bryant Brook	Lower Ballard Pond	Tributary E to Beaver Lake
Cohas Brook	Lower Beaver Lake	Tributary E to Little Cohas Brook
Country Pond	Meadow Pond	Tributary F to Beaver Lake
Cunningham Brook	Nesenkeag Brook	Tributary G to Beaver Brook
Drew Brook	Nudds Canal	Tributary H to Drew Brook
Dudley Brook	Pickering Brook	Tributary H to Nesenkeag Brook

TABLE 2 - FLOODING SOURCES STUDIED BY DETAILED METHODS - continued

Exeter River	Piscassic River	Tributary J to Black Brook
Flatrock Brook	Piscataqua River	Tributary O to Beaver Brook
Golden Brook	Policy Brook	Tuxbury Pond
Grassy Brook	Porcupine Brook	Upper Ballard Pond
Great Bay	Porcupine Brook Tributary	Upper Beaver Brook
Great Pond	Powwow Pond	Wash Pond
Hornes Brook	Powwow River (Downstream Reach)	Wash Pond Tributary
Hill Brook	Powwow River (Upstream Reach)	West Channel Policy Brook
Hog Hill Brook	Shields Brook	Winnicut River
Hidden Valley Brook	Shop Pond	World End Brook
Island Pond	Spicket River	World End Pond
Kelly Brook		

The 2005 countywide FIS also incorporated the determinations of letters issued by FEMA resulting in map changes (Letter of Map Revision [LOMR], Letter of Map Revision- based on Fill [LOMR-F], and Letter of Map Amendment [LOMA]), as shown in Table 3, "Letters of Map Change."

TABLE 3 – LETTERS OF MAP CHANGE

Community Name	Flooding Source(s)/ Project Identifier	Effective Date	Type
Portsmouth, City of	Pickering Brook/Ocean Road Development Corporation project	October 6, 1999	LOMR
Rye, Town of	Atlantic Ocean/Brown Property shore protection project	February 15, 2001	LOMR
Salem, Town of	West Channel Policy Brook/Powers Builders property	September 15, 1999	LOMR
Epping, Town of	Lamprey River/downstream of Prescott Road bridge	September 7, 1993	BADL

The areas studied by detailed methods were selected with priority given to all known flood hazard areas and areas of projected development and proposed construction.

Numerous flooding sources in the county were studied by approximate methods. Approximate analyses were used to study those areas having a low development potential or minimal flood hazards. The scope and methods of study were proposed to, and agreed upon by, FEMA and the communities in Rockingham County.

For the 2005 countywide study, several areas of approximate flooding were extended to match the approximate flooding across community corporate limits within Rockingham County and across the county boundary from contiguous counties. The delineation involved the use of topographic maps at a scale of 1:24,000 and contour intervals of 10 and 20 feet (U.S. Department of Interior, 1966).

Three “Little Rivers” exist in Rockingham County. For clarification purposes, they have been renamed in the FIS as follows: Little River in the Town of Exeter is Little River No. 1; Little River in the Town of North Hampton is Little River No. 2; Little River in the Town of Plaistow is Little River No. 3. In addition, Tributary D in the Town of Londonderry has been renamed in the FIS as Tributary O to Beaver Brook.

The January 29, 2021 Countywide Revision

The January 29, 2021 countywide revision consisted of revisions to the coastal and riverine analyses in 14 contiguous communities located in eastern Rockingham County. These communities include: Exeter, Greenland, Hampton, Hampton Falls, the Village District of Little Boar’s Head, New Castle, Newfields, Newington, Newmarket, North Hampton, Portsmouth, Rye, Seabrook, Seabrook Beach Village District, and Stratham.

The work performed in these communities consisted of revisions as follows:

- New Atlantic coastal analysis
- Revised Zone AE studies on the Exeter and Lamprey Rivers
- Revisions due to updated topographic data on the Piscataqua River, Great Bay shoreline, Squamscott River, Little River No. 1 (in Exeter), Little River No. 2 (in North Hampton), Pickering Brook, Piscassic River, and the Winnicut River
- New model-backed Zone A studies replaced all existing Zone A streams

The updated topographic data used for the 2013 study was based on LiDAR collected at a 2.0 meter nominal post spacing (2.0m GSD) for approximately 8,200 mi^2 of coastal areas including parts of Maine, New Hampshire, Massachusetts, Rhode Island, Connecticut, and New York, as part of the American Recovery and Reinvestment Act (ARRA) of 2009. The data was collected by Photo Science Inc. in May of 2011. No snow was on the ground and rivers were at or below normal levels. Some areas of the project required 1.0 meter nominal post spacing (1.0m GSD), and a required 9.25cm Vertical Accuracy. The study area was covered by 1.0 meter post spacing LiDAR data and a portion of the contributing drainage area was covered by the 2.0 meter post spacing LiDAR data. A seamless Digital Elevation Model (DEM) at a 10 ft resolution was created combining the above datasets to create a base elevation for the coastal analyses.

For the Isles of Shoals, the LiDAR was available for the northern portion of Star Island only. For the remainder of Star Island, a topographic map with 2-foot contour intervals developed by Ambit Engineering, Inc, in May of 2011, based on information collected in 1916, was digitized and converted to NAVD 88. The topography for the other islands, which are low lying, was taken from USGS 10 meter digital elevation models.

No Letters of Map Revision (LOMRs) were incorporated in the 2013 coastal update.

2.2 Community Description

Rockingham County is located in southeastern New Hampshire. In Rockingham County, there are 37 communities. The Towns of Northwood, Nottingham, and Deerfield are located in the northwestern section of the county. The Towns of Epping, Newmarket, and Newfields are located in the northern section of the county. In the eastern part of the county, lie the City of Portsmouth and the Towns of Newington, Greenland, New Castle, Stratham, Exeter, North Hampton, and Rye. The Seabrook Beach Village District, Village District of Little Boar's Head, and the Towns of Hampton, Hampton Falls, and Seabrook are located in the southeastern part of the county. The Towns of Brentwood and Fremont are located in the center of Rockingham County. In the southern section of the county lie the Towns of Sandown, Danville, Kingston, East Kingston, Kensington, Hampstead, Atkinson, Plaistow, Newton, and South Hampton. In the southwestern section of the county, the Towns of Derry, Londonderry, Windham, and Salem are located. The Towns of Candia, Raymond, Auburn, and Chester are located in the western part of Rockingham County.

Rockingham County is bordered to the north by communities of Strafford County: the Towns of Strafford, Barrington, Lee, Durham, and Dover. To the northeast, the county is bordered by communities of York County, Maine: the Towns of Kittery and Eliot. It is bordered to the northwest by communities of Merrimack County: the Towns of Pittsfield, Epsom, Allenstown, and Hooksett. Rockingham County is bordered to the southwest by communities of Hillsborough County: the City of Manchester and the Towns of Bedford, Merrimack, Litchfield, Hudson, and Pelham. To the south, the county is bordered by the communities of Essex County, Massachusetts: the Cities of Methuen and Haverhill and the Towns of Amesbury and Salisbury. According to the U.S. Census Bureau, the population of Rockingham County was 295,223 in 2010.

The topography of the county is flat coastal plains to the east, gently rolling hills to the south and center, and more hilly terrain to the northwest. The Atlantic coast is characterized by sandy beaches, rocky headlands, wetlands, and offshore reefs and ledges. The development in Rockingham County is primarily residential.

The climate of the county can be classified as modified continental. The average annual temperature is approximately 47 degrees Fahrenheit (U.S. Department of Commerce). The average rainfall of the county is 42 inches per year (FEMA, 1993).

The main flooding sources in Rockingham County are the Atlantic Ocean to the east, Exeter River in the east, Lamprey River in the center, Little Cohas Brook in the west, and Beaver Brook in the south.

2.3 Principal Flood Problems

Past history within the county indicates that major floods occur during the spring, fall, and winter seasons. Some of the most severe flooding occurs in early spring as a result of snowmelt and heavy rains in conjunction with ice dams. Less frequently, flooding occurs later in the year as a result of localized thunderstorms or hurricanes. The largest of these floods occurred in March 1896, March 1936, March 1977, January 1978, March 1983, April 1987, July 1934, March 1936, and April 1987. No estimate of peak flow was available for the 1896 flood, but the 1936, 1977, and 1987 flows were estimated at 5,490, 5,000, and 7,500 cfs, respectively.

Low-lying areas are subject to periodic flooding caused by overflows of the Lamprey River, Exeter River, and Squamscott River. The most severe flooding occurs in early spring as a result of snowmelt and heavy rains. In the past, portions of Prescott Road along Lamprey River have flooded nearly every year. The 1989 replacement of the Prescott Road Bridge over the Lamprey River should help alleviate this condition. During the April 1987 flood, up to two feet of water covered portions of Harriman Hill Road. Old Manchester Road and Main Street were also affected by flooding of the Lamprey River in 1987.

The low-lying areas along the Atlantic coast are subject to the periodic flooding and wave attack that accompany northeasters and hurricanes. The majority of these storms cause damage only to low coastal roads, boats, and seawalls. Occasionally, a major storm accompanied by strong onshore winds and high tides results in surge and wave activity that cause extensive property damage and erosion. Some of the more significant storms include those of December 1909, December 1959, February 1972, and February 1978. The recurrence intervals for these storms were 160 years, 15 years, 10 years, and 70 years, respectively. Other significant storms occurred in the vicinity of North Hampton in November 1945, November 1963, November 1968, and November 1969. These storms damaged harbors, marinas, and commercial and residential developments along the flood-prone coastline (FEMA, City of Portsmouth, 1981). Other more recent noteworthy storms causing significant flooding in the area have included May 2006, April 2007, and March 2010.

During spring runoff periods, the Exeter River frequently flooded roads on the south side of the Town of Exeter, including Court Street, Crawford Avenue, and Portsmouth Avenue. A USGS surface-water discharge station was active on the Exeter River at the Haigh Road Bridge in Brentwood during a 1996 storm and recorded a peak discharge of 3,060 cfs. This event had a recurrence interval of approximately 100 years. Additional areas were flooded by the Exeter River, due to rainfall associated with hurricanes in 1938 and 1954. The area on the north side of the Exeter River in Tib's Grove is subject to occasional backwater flooding from Phillips dam in the Town of Brentwood.

The major portion of the Spicket River floodplain lies between the Arlington Mill Reservoir and the Massachusetts State line. Because of its flat gradient and the numerous swamps and lakes in the watershed, peak flows and stages on the Spicket River are a function of high-volume rainfall.

The middle reach of Policy Brook between Rockingham Park Boulevard and Pleasant Street is subject to periodic flooding due to its flat gradient and the many restrictions caused by inadequately sized pipes and culverts.

The Squamscott River periodically floods the Swasey Parkway and other low-lying areas during unusually high tides. In the past, within the Town of Greenland, little significant damage has occurred in these areas, however, due to the general absence of buildings and other structures.

Low-lying areas adjacent to Great Bay are subject to periodic flooding. Little significant damage occurs in these areas, however, due to the general absence of buildings and other structures.

Areas along Pickering Brook are subject to flooding. Present damage potential is slight due to absence of structures in affected marshes. However, future flood damage could be significant if development upstream of State Route 151 is allowed to lower the road elevation of 31 feet. This road crest is the emergency spillway necessary if debris clogs the only culvert through the dam-like road fill. The extensive upstream beaver action and by-products of urbanization could be sources of flood-creating debris.

Extensive flooding in the low-lying areas surrounding the Powwow Pond system occurred in March 1983. During the flood, elevations on Great Pond peaked at approximately 2 feet above the dam crest. According to records at the New Hampshire Department of Water Resources, this is the maximum recorded elevation for Great Pond.

Minor damage to Cuba Road frequently occurs due to flooding of the Piscassic River. This flooding usually occurs during March and April during spring rains and snowmelt. Floods occurring during other seasons are often associated with debris clogging culverts. Due to the natural and manmade hydraulic structures along the Piscassic River, and the number of beavers in the watershed, collection of debris generally compounds flooding.

Flooding problems have occurred in the past and may be expected to occur in the future at the undersized culvert at State Route 125 crossing of Kelly Brook. Such situations can create backwaters of depth sufficient to inundate extensive areas of land.

2.4 Flood Protection Measures

The State of New Hampshire provides concrete seawalls and stone revetments to protect coastal highways. The USACE built shoreline protection structures at Wallis Sands State Beach (U.S. Department of the Interior, 1962) and at Hampton Beach (New England River Basins Commission, 1980). The Town of Rye maintains a small portion of the waterfront barrier in the southern end of town. Other protective coastal structures were constructed and are maintained by the local municipalities and private property owners to satisfy their individual requirements and financial capabilities. These structures include such backshore protection as timber and steel sheet piles, bulkheads, stone revetments, concrete seawalls, and pre-cast concrete units (U.S. Army Corps of Engineers, 1971). Limited financial resources sometimes result in less than adequate protection.

A breakwater located in the Town of Rye that is maintained by the USACE provides some protection for Little Harbor. There are some small-scale protective structures maintained by private homeowners that satisfy individual requirements.

A protective breakwater is located on the north shore of the Hampton Harbor inlet. It extends approximately 1,000 feet southeast into the Atlantic Ocean and protects the mouth of both Hampton and Seabrook Harbors from wave action.

The Water Division of the New Hampshire Department of Environmental Services controls the Trickling Falls Dam at the outlet of Powwow Pond and the dam at the outlet of Great Pond. During the fall and early winter, flash boards are removed from these dams and the ponds are lowered to provide extra storage capacity for spring runoff. There are also extensive low-lying areas surrounding the Powwow Pond system. These areas provide natural storage that serves to reduce flood peaks.

Dams at the outlet of Powwow Pond and Great Pond in East Kingston provide some flood protection in areas upstream of South Hampton; however, the effect on peak discharge in South Hampton is not significant (U.S. Department of the Interior, 1962). Likewise, the dam at Tuxbury Pond provides negligible flood protection.

In the Town of Stratham, zoning has been established to prevent development within 150 feet of the Squamscott River and 100 feet of major freshwater streams.

There is a levee separating sewage treatment plant stabilization lagoons from the Squamscott River. FEMA specifies that all levees must have a minimum of 3 feet freeboard against 1 percent annual chance flooding to be considered a safe flood protection structure. The levee has a nominal crest elevation of 14 feet, yielding a 6-foot freeboard which meets FEMA freeboard requirements. There are also several small dams within the town. However, they do not significantly alter flood flows.

The numerous swampy areas and small ponds within Rockingham County provide natural storage that serves to reduce flood peaks.

Newmarket has no existing or proposed flood control structures. During extreme flood events, floodwaters from the Lamprey River overflow State Route 108 upstream in Durham and are diverted into the Oyster River basin. These overflows or diversions reduce peak flood discharges of the Lamprey River before it reaches the Town of Newmarket. During a 1 percent annual chance flood, diversions to the Oyster River basin reduce flood peaks in Newmarket by approximately 20 percent (FEMA, 1991).

3.0 ENGINEERING METHODS

For the flooding sources studied in detail in the county, standard hydrologic and hydraulic study methods were used to determine the flood hazard data required for this FIS. Flood events of a magnitude which are expected to be equaled or exceeded once on the average during any 10-, 50-, 100-, or 500-year period (recurrence interval) have been selected as having special significance for floodplain management and for flood insurance rates. These events, commonly termed the 10-, 50-, 100-, and 500-year floods, have a 10-, 2-, 1-, and 0.2-percent chance, respectively, of being equaled or exceeded during any year. Although the recurrence interval represents the long term average period between floods of a specific magnitude, rare floods could occur at short intervals or even within the same year. The risk of experiencing a rare flood increases when periods greater than 1 year are considered. For example, the risk of having a flood which equals or exceeds the 100-year flood (1-percent chance of annual exceedance) in any 50-year period is approximately 40 percent (4 in 10), and, for any 90-year period, the risk increases to approximately 60 percent (6 in 10). The analyses reported herein reflect flooding potentials based on conditions existing in the county at the time of completion of this FIS. Maps and flood elevations will be amended periodically to reflect future changes.

3.1 Riverine Hydrologic Analyses

Hydrologic analyses were carried out to establish the peak discharge-frequency relationships for the flooding sources studied in detail affecting the county.

For each community within Rockingham County that has a previously printed FIS report, the hydrologic analyses described in those reports have been compiled and are summarized below.

Pre-countywide Analyses

Discharge-frequency data for the flooding sources studied by detailed methods were determined from equations based on multiple-regression analyses of data from USGS gaged sites in New Hampshire and adjacent areas of bordering states (U.S. Department of the Interior, 1978). The equations contain the independent variables basin drainage area, main-channel slope, and a precipitation intensity index.

No stream gages have been operated in the Powwow River Basin. To calculate the 1 percent annual chance frequency flood discharges, three separate reports were consulted (U.S. Department of the Interior, 1975; U.S. Department of the Interior, 1978; and U.S. Department of the Interior, 1983). The three reports document techniques that can be used to estimate flood peaks on rural basins in Maine, New Hampshire, and Massachusetts. In each of the reports, regression equations were used to relate flood-peak discharges to basin characteristics such as drainage area, stream slope, basin storage, and precipitation. The Powwow River basin is located near coastal New Hampshire in an area close to both Massachusetts and Maine. Data from this portion of New Hampshire was included in each of three studies and as a result, information from all of the reports could be appropriate for use.

Flood discharges were computed using equations from each of the three reports and the results were carefully reviewed. Analysis indicated that use of the equation documented in the report for Massachusetts would be most appropriate (U.S. Department of the Interior, 1983). The Massachusetts report is the most current of the three and it used a larger data base. Most importantly, the area studied in the report was divided into three separate regions and regression equations were calculated for each. One of the three zones was the eastern or coastal area, the region in which the Powwow River basin is located. Regression equations developed for the eastern region were specific to the coastal type of watershed. The Massachusetts equations have also been used in two other studies in the Powwow River basin: East Kingston, New Hampshire, and Amesbury, Massachusetts (FEMA, April 1986; FEMA, 1982).

Due to the excessive amount of natural storage in the Powwow Pond system, adjustment of the peak discharge was required. Using techniques documented in a USGS report, a basin lag time and an inflow hydrograph were computed with a peak discharge of 1,240 cfs (U.S. Department of the Interior, 1983). The resultant hydrograph was routed through the Powwow Pond system using the Modified Puis Method (Linsley, R. K., et al., 1982). The Modified Puis method is based on a form of the continuity equation in which for any time period, average inflow less average outflow equals change in storage within the system. Based on this analysis, the resultant 1 percent annual chance flood frequency outflow from Powwow Pond is 850 cfs. Drainage area ratios were used to compute 1 percent annual chance flood frequency peak discharges at alternate points in the Powwow Pond system as a function of the outflow from Powwow Pond.

Due to the absence of gaged data, the principal source of data for defining discharge-frequency relationships for all detailed streams in Windham (Beaver Brook, Golden Brook, Flatrock Brook, and Hidden Valley Brook) was regional discharge-frequency equations developed by Manuel Benson. These regional equations relate topographical and precipitation characteristics to streamflow (U.S. Department of the Interior, 1962).

The Squamscott River, Exeter River, Little River No. 1, Little River No. 2, and Winnicut River are ungaged. The 10-, 2-, and 1-percent annual chance flood discharges were based on regional peak discharge and frequency formulas developed by the USGS (U.S. Department of the Interior, 1978). A separate evaluation of these formulas was performed and found to be applicable to the Exeter region. In addition, the formulas were expanded and an equation was developed to predict the 0.2 percent annual chance flood discharge. The USGS formulas predict discharges based on the parameters of watershed drainage area, main channel slope, and rainfall intensity.

Hydrologic analysis of the 1 percent annual chance flood was performed for Dudley Brook. Discharge for the 1 percent annual chance flood was based on a U.S. Water Resources Council log- Pearson Type Ill frequency analysis of gage data at the USGS gage no. 01073600 on Dudley Brook near the Town of Exeter, which has 23 years of record (1962 -1985) and a drainage area of 12.1 square miles (U.S. Water Resources Council, 1976). Discharges from the gage analysis were transferred to stream stations removed from the gage by the formula:

$$Q / Qg = (A/Ag)^{0.75}$$

Where Q is the discharge at the different specific site locations, Qg is the discharge at the USGS stream gage, and A and Ag are the drainage areas at the specific site and at the USGS stream gage, respectively.

Discharges for the Little River No. 3, Kelly Brook, and Bryant Brook were developed by combining the results of regional flood frequency equations with discharge values transposed from gaged basins in the region, which are similar in size and characteristics, to those studied. The regional equations, developed from regression analysis of gaging records for eastern Massachusetts using basin parameters to estimate flood peaks, were applied at several points along each stream (U.S. Geological Survey, 1977). USGS gage no. 0107300 on the Oyster River in Durham was used to transpose discharges to the Little River No.3. This gage has a period of record of 43 years and a drainage area of 12.1 square miles. The USGS gage no. 01073600 on Dudley Brook near Exeter was used to transpose discharges to Kelly Brook and Bryant Brook. The transposition was carried out using the formula as shown above.

The principal sources of data for defining discharge-frequency relationships for detailed study streams in Londonderry (Beaver Brook, Black Brook, Cohas Brook, Little Cohas Brook, Nesenkeag Brook, Shields Brook, Tributary C to Beaver Brook, Tributary E to Little Cohas Brook, Tributary H to Nesenkeag Brook, Tributary J to Black Brook, Tributary 0 to Beaver Brook, and Upper Beaver Brook) were the regional equations developed by Manuel Benson of the USGS. These regional equations relate topographical and precipitation characteristics to stream flow (U.S. Department of the Interior, 1962).

Discharges for Hidden Valley Brook were derived by comparing values predicted by regional equations and discharge-frequency relationships based on a log-Pearson Type III analysis (U.S. Water Resources Council, 1976) for the gages in the vicinity on Stony Brook (USGS Gage No. 093800) and on Dudley Brook (USGS Gage No. 073600) (U.S. Department of the Interior, 1976).

Discharge-frequency data for Hog Hill Brook, Wash Pond Tributary, Hill Brook, Wash Pond, and Shop Pond were determined from equations based on multiple-regression analyses of data from USGS gaged sites in New Hampshire and adjacent areas bordering states (U.S. Department of the Interior, 1978). The equations contain the independent variable basin drainage area, main-channel slope, and a precipitation intensity index.

Discharge values for the Exeter River in the Town of Brentwood were obtained from the previous FISs for the Towns of Brentwood and Exeter (FEMA, 1980; FEMA, May 1982). Peak discharges for the Exeter River were obtained from the Town of Exeter FIS, enacted on November 17, 1981, and were based on regional peak discharge and frequency formulas developed by the USGS and expanded to predict the 0.2 percent annual chance flood discharge (U.S. Department of the Interior, 1978). Peak discharges for the Exeter River obtained from the original FIS for the Town of Brentwood were based on a flow rate per unit area relationship with a USGS surface-water discharge station on the Lamprey River (FEMA, 1981).

For the Exeter River in the Town of Raymond, only the peak 1 percent annual chance flood return period discharge was computed. The peak discharge at the Blueberry Hill Road bridge was available from NHDOT (U.S. Department of the Interior, 1962). The value was computed using regionally developed peak flows for more frequent storms in combination with a methodology involving a probability distribution to produce the 1 percent annual chance flood peak discharge. The peak 1 percent annual chance flood discharge computed by Rivers Engineering Corporation using methodology used as part of the FISs for other New Hampshire communities was not significantly different from the value computed by the NHDOT (U.S. Water Resources Council, 1977). The NHDOT value was adjusted to other location on the Exeter River based on the ratio of the drainage areas.

Gaging stations on the Lamprey River, located approximately 9 miles north of the Exeter River, and on Dudley Brook, a tributary of the Exeter River, were the principal sources of data for determining discharge-frequency relationships for the Exeter River in the Town of Fremont. The gages have been in operation since 1934 and 1962, respectively. Values for the 10-, 2-, 1-, and 0.2-percent annual chance flood peak discharges were obtained from a log-Pearson Type III distribution of annual peak flow data.

Flows for the various frequencies were transformed to a flow rate per unit area and plotted versus drainage area on log-log paper. A straight line was drawn through the pairs of flow-drainage area coordinates computed for the gages. Flows for drainage areas of the Exeter River at various locations in Fremont were taken from the plot.

A check on the procedure described above was made at the Fremont-Brentwood corporate limits by application of regional relationships developed in USGS Water-Supply Paper 1580-B and Water Resources Investigations 78-47 (U.S. Department of the Interior, 1962; U.S. Department of the Interior, 1978). The regression analyses developed in these reports relate peak discharge to drainage area, channel slope and rainfall intensity. The method in Water-Supply Paper 1580-B also considers indices for surface water area, January temperature, and orographic effect.

Since the Piscassic River is ungaged, discharge-frequency data for this flooding source was developed using the USGS Water Resources Investigation Report, WRI 78-47, a synthetic runoff procedure that relies on regionalized climatological data coupled with the individual stream physical characteristics for input (U.S. Department of the Interior, 1978).

For Beaver Brook, Cunningham Brook, Drew Brook, Taylor Brook, Tributary E to Beaver Lake, Tributary F to Beaver Lake, Tributary G to Beaver Lake, Tributary H to Drew Brook, and Tributary O to Beaver Brook, the principal source of data for defining discharge-frequency relationships was the regional discharge-frequency equations developed by the USGS (U.S. Department of the Interior, 1962). These regional equations relate topographical and precipitation characteristics to streamflow. Due to the extensive upstream channel and pond storage and flatter slopes, discharges for the Homes Brook-Shields Brook watershed were derived using a regional discharge-frequency equation based on streams with similar characteristics (U.S. Department of the Interior, 1974).

Discharges for Beaver Brook were modified due to the storage effects of Beaver Lake. Golden Brook was modified due to the storage effects of Cobbetts Pond and Moeckel (Simpson)-Rock Ponds. Taylor Brook was modified due to the storage effects of Ballard Pond. A reservoir routing using a numerical iteration method (Viessman, Warren J., et al., 1972) was performed on Beaver Lake and Island Pond. The results of this routing were used to adjust the discharges of Beaver Brook and Taylor Brook and to establish the water-surface elevations of Beaver Lake for the 10-, 2-, 1-, and 0.2-percent annual chance floods. The results of the reservoir routing performed on Cobbetts Pond were used in conjunction with the results of Benson's equation to adjust the discharges of Golden Brook between Tributary C and Moeckel (Simpson)-Pond. Below Moeckel (Simpson) Pond, the discharges were adjusted using the results of the reservoir routing performed on Moeckel (Simpson)-Rock Ponds.

The principal source of data for defining the discharge-frequency relationships for the Lamprey River was the USGS gaging station located in Durham, which had been operating since 1934. Values of the 10-, 2-, 1-, and 0.2-percent annual chance flood peak discharges were obtained from a log-Pearson Type III distribution of annual peak flow data (U.S. Department of the Interior, 1967).

Discharge-frequency estimates for areas above the stream gage were developed using a regional relationship developed in a USGS report (U.S. Department of Agriculture, 1979). The regression analysis developed in this report relates peak discharge to drainage area, channel slope, rainfall intensity, surface storage, January temperature, and orographic influences. The flow estimates developed by the USGS were estimated by multiplying the ratio of discharge based on gage data to that based on the USGS method for the gaged area time the discharge developed by the USGS at locations within Raymond.

Flood flows for the Lamprey River were determined by using regional equations for peak discharges applicable to the area (Southeastern New Hampshire Regional Planning Commission, 1974). This method combines basin and climatic characteristics through specific regression equations to yield discharges for the 10-, 2-, and 1-percent annual chance floods. Peak discharges for the 0.2 percent annual chance flood return period storm were based on an equation developed as an extension of the methodology developed by the USGS and used for prediction of the peak 0.2 percent annual chance flood return period discharge as part of the FISs for other New Hampshire communities (U.S. Water Resources Council, 1977; Southeastern New Hampshire Regional Planning Commission, 1974). Peak flows computed by use of the regional equations were determined to be more appropriate for the Lamprey River in Raymond than a transposition of peak flows computed at the gaging station downstream in Durham. As described below, the transposition of flows from the gage produced peak flows in Raymond that did not adequately reflect the magnitude of flooding experienced by the community.

There are no continuous records of discharges on the Spicket River. A peak discharge for the March 1968 flood was computed and reported by the USGS for the Spicket River at a dam located approximately 1.5 miles below the Salem, New Hampshire-Methuen, Massachusetts, town line. A peak discharge of 1,440 cubic feet per second (cfs) was computed at this site, which has a total drainage area of 73.8 square miles.

A gaged stream in the region with similar hydrologic characteristics is the Parker River, located approximately 15 miles southeast of Salem. This river has 30 years of discharge records for a contributing watershed of 21.6 square miles. Discharge frequencies for the Spicket River were estimated using peak discharge frequency data for the Parker River. Frequencies for the Parker River were developed from historical flow data using the log-Pearson Type Ill statistical distribution (U.S. Water Resources Council, 1976, Bulletin 15). The frequencies for the Spicket River were then developed by multiplying the Parker River flows by the ratio of the known 1968 peak discharges on both streams. Discharges at other locations along the Spicket River were derived by multiplying the adopted discharges at the dam in Methuen by a factor equal to the ratio of the drainage areas to the 0.7 exponential power.

Over the years, Policy Brook has been modified by the installation of two long conduits under and adjacent to Rockingham Park. Conduit A extends from just upstream of Pleasant Street to just above the brook's second crossing of the Boston and Maine Railroad and State Route 28. It passes under the horse barn area of the race track. Conduit B and an excavated section of open ditch run along the railroad and bypass the second railroad/State Route 28 crossing. This bypass was installed to reduce the flooding of a mobile home park just to the east of State Route 28.

The installation of the bypass results in Policy Brook having two channels, an East Channel and a West Channel in this area. The West Channel (conduit-ditch) carries all of the flows from upper Policy Brook during non-flood periods as the second railroad/State Route 28 crossing has been partially blocked.

Flood discharges for the lower reaches of Policy Brook, its East Channel, and Unnamed Brook were developed by estimating the mean annual peak flows based on an appraisal of existing culvert size on the streams and the sluggish hydrologic character of the watersheds. Rarer flood flows for the brooks were determined as multiples of the mean annual flows by use of the "Bigwood-Thomas" type flood formula as well as by rainfall frequency comparisons (U.S. Geological Survey, 1955). Both the Technical Release No. 20 (TR-20) and the Technical Release No. 55 (TR-55) models were used to develop the 1 percent annual chance flood discharges at various points in the watershed (U.S. Department of Agriculture, 1992; U.S. Department of Agriculture, 1986). TR-20 is a synthetic rainfall runoff procedure that relies on regionalized climatological data coupled with the individual stream physical characteristics for input (U.S. Department of Agriculture, 1983). Drainage areas, land uses and times of concentration were computed using USGS quadrangle coverage. A rainfall of 6.5 inches in a 24-hour period was used to produce the unit hydrographs.

The peak discharge for the April 1987 flood at the USGS gage at Packers Falls was 7,500 cfs. The 1 percent annual chance flood discharge at the gage was determined in Section 3.1 to be 7,300 cfs. The 1987 flood was therefore slightly greater than the 1 percent annual chance flood. Peak flood elevations that occurred during the 1987 flood were identified and surveyed in the field by the study contractor. The 1 percent annual chance flood profile for Lamprey was based on these elevations and data available for Durham (FEMA, 1991).

A TR-55 analysis was used to develop discharges on Porcupine Brook and Porcupine Brook Tributary.

For the analysis of the West Channel and the upper reaches of Policy Brook, temporary flood storage in Canobie Lake, in the large, flat area between Pleasant Street and South Policy Road and in Rockingham Park at the outlet of Conduit A were included in the TR-20 model. The area above Pleasant Street, because of its size and the limited capacity of Conduit A, is especially effective in reducing flood flows.

Since Pickering Brook is not gaged, discharge-frequency data for this stream were developed using TR-20.

For World End Pond, both the outlet channel and the constricted downstream road crossings (Lawrence Road and Farm Road) were modeled. For the 1 percent annual chance flood, the road crossings were found to control the upstream water levels and these stage discharge relationships were used in the TR-20 model.

Only the 1 percent annual chance flood elevations have been determined for stillwater elevations for Wash Pond, Country Pond, Great Pond, Piscataqua River, World End Pond, and Shop Pond. No adjustments to computed "Stillwater Elevations" were made to account for changes in storage in Wash Pond and Shop Pond. These changes in storage were considered insignificant.

Discharges for approximate study streams were also developed using Manuel Benson's regional discharge-frequency equations (U.S. Department of the Interior, 1962).

2005 Countywide Analyses

No hydrologic analyses were conducted for the 2005 countywide study.

The January 29, 2021 Countywide Revision

For the January 29, 2021 countywide revision, hydrologic analyses were carried out to establish peak discharge-frequency relationships for each flooding source studied by approximate methods in the communities studied, and for the flooding sources studied in detail affecting the towns of Exeter and Newmarket. Discharges for the 1-percent-annual-chance recurrence interval for all approximate study streams in these communities were determined using regression equations found in Olson, S.A., 2009, Estimation of flood discharges at selected recurrence intervals for streams in New Hampshire, U.S. Geological Survey Scientific Investigations Report 2008-5206.

Hydrologic analyses for the Lamprey River (Newmarket, NH) was based on a log-Pearson Type III frequency analysis of the stream gage data at the USGS stream gage no. 01073500 at Packers Falls at Durham, NH which has 77 years of record (1934 – 2011) and a drainage area of 185 square miles. Based on a recently completed Lamprey River watershed study at the University of New Hampshire (Scholz, 2011), it was assumed that 20% of Lamprey River flood flow is diverted to the Oyster River watershed via La Roche and Longmarsh Brooks.

Discharges from the stream gage analysis were transferred to stream locations removed from the stream gage by the formula:

$$Q/Q_g = (A/A_g)^{1.0}$$

Where Q is the discharge at the different specific site location, Q_g is the discharge at the USGS stream gage, and A and A_g are the drainage areas at the specific site and at the USGS stream gage, respectively.

A summary of the drainage area-peak discharge relationships for all of the streams studied by detailed methods is shown in Table 4, "Summary of Discharges."

TABLE 4 – SUMMARY OF DISCHARGES

Flooding Source and Location	Drainage Area (sq. miles)	Peak Discharges (cfs)			
		10% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
BEAVER BROOK					
At Pelham-Windham corporate limits	51.0	1,500	2,560	3,180	4,930
At Pelham-Windham-Hudson corporate limits	48.6	1,450	2,470	3,070	4,750
Downstream of Robinson Pond Brook	48.3	1,400	2,430	3,010	4,670
Upstream of Robinson Pond Brook	45.0	1,310	2,360	2,900	4,490
At Londonderry-Windham-Hudson corporate limits	44.2	1,200	2,120	2,800	4,150
At confluence with Black Brook	38.3	1,040	2,100	2,580	4,050
Upstream of Tributary C to Beaver Brook near Station 20.5	32.7	860	1,760	2,160	3,600

TABLE 4 - SUMMARY OF DISCHARGES – continued

Flooding Source and Location	Drainage Area (sq. miles)	Peak Discharges (cfs)			
		10% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
BEAVER BROOK (continued)					
From upstream of Tributary C to Beaver Brook in Londonderry to downstream of Tributary O to Beaver in Derry[1]	32.7[2]	800	1,660	2,050	3,500
From upstream of Tributary O to Beaver Brook to downstream of Hornes Brook[1]	24.3[2]	750	1,520	1,860	3,300
At Londonderry-Windham-Derry corporate limits	27.0	720	1,510	1,860	3,300
From upstream of Hornes Brook to downstream of Tributary G to Beaver Brook[1]	17.5[2]	400	1,150	1,440	2,880
At Londonderry-Derry corporate limits	26.3	720	1,510	1,860	3,300
From upstream of Tributary G to Beaver Brook to downstream of Tributary B to Beaver Brook	12.5[2]	130	510	650	1,410
From upstream of Tributary B to Beaver Brook to 650 feet downstream of outlet of Beaver Lake[1]	12.0[2]	65	380	430	960
At outlet of Beaver Lake	11.2	32	240	320	730
BLACK BROOK					
At mouth	5.6	185	345	425	830
At Adams Road	2.0	20	60	90	290
BRYANT BROOK					
Downstream limit of detailed study	3.9	175	290	355	550
COHAS BROOK					
At Londonderry-Manchester corporate limits	12.3	410	760	990	1,550

[1]Reach Discharge
[2]Drainage area at downstream limit of reach

TABLE 4 - SUMMARY OF DISCHARGES – continued

Flooding Source and Location	Drainage Area (sq. miles)	Peak Discharges (cfs)			
		10% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
CUNNINGHAM BROOK					
At confluence with Leavitt and Drew Brooks	3.4	245	630	775	1,540
At confluence with Tributary H to Nesenkeag Brook	2.0	145	390	480	1,000
At Hampstead Road	1.1	75	215	260	560
DUDLEY BROOK					
At eastern corporate limits of town of Brentwood	6.1	*	*	589	*
At USGS gaging station 01073600	5.0	*	*	506	*
DREW BROOK					
From Island Pond to confluence of Leavitt and Cunningham Brooks[1]	5.0[2]	115	285	350	700
EXETER RIVER					
Downstream of the confluence of Little River No. 1	114.6	2,811	4,107	4,827	6,518
Upstream of the confluence of Little River No. 1	100.8	2,453	3,589	4,219	5,704
Upstream of confluence of Great Brook	89.9	2,173	3,183	3,741	5,064
At eastern corporate limits of the Town of Brentwood	73.0	1,990	2,880	3,280	4,230
At Haigh Road	64.0	1,810	2,640	3,010	3,900
At eastern corporate limits of the Town of Fremont	60.0	1,740	2,520	2,880	3,750
At downstream corporate limits of the Town of Raymond	49.6	*	*	2,700	*
At Blueberry Hill Road bridge	46.8	*	*	2,550	*
At upstream corporate limits of the Town of Raymond	37.1	*	*	2,020	*

[1]Reach Discharge

[2]Drainage area at downstream limit of reach

*Data not available

TABLE 4 - SUMMARY OF DISCHARGES – continued

Flooding Source and Location	Drainage Area (sq. miles)	Peak Discharges (cfs)			
		10% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
FLATROCK BROOK					
At inlet to Shadow Lake	7.3	270	640	760	1,450
Downstream of tributary near Station 0.9	6.9	220	540	640	1,230
Upstream of tributary near Station 0.9	5.9	190	460	550	1,030
At outlet to Seavey Pond	5.3	170	420	495	960
GOLDEN BROOK					
At outlet to Moeckel (Simpson)-Rock Ponds	11.5	100	550	750	1,490
At inlet to Moeckel (Simpson)-Rock Ponds	10.5	340	805	960	1,700
At downstream confluence with Tributary B	5.9	273	665	791	1,400
At upstream confluence with Tributary B	3.1	142	369	439	860
At downstream confluence with Tributary A	2.4	103	273	325	630
GRASSY BROOK					
At confluence with Powwow River	1.67	*	*	198	*
HIDDEN VALLEY BROOK					
At confluence with Beaver Brook	2.5	150	270	325	540
At culvert near station 1.0	1.9	120	220	260	430
At Londonderry Road culvert	1.1	75	135	165	275

*Data not available

TABLE 4 - SUMMARY OF DISCHARGES – continued

Flooding Source and Location	Drainage Area (sq. miles)	Peak Discharges (cfs)			
		10% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
HILL BROOK					
At State Route 111	1.52	*	*	120	*
HOG HILL BROOK					
At Haverhill Road	8.38	*	*	680	*
At Kathi Lane	5.52	*	*	410	*
At Island Pond Road in the Town of Atkinson	4.75	*	*	380	*
HORNES BROOK					
From Beaver Brook to Hornes Pond[1]	6.82	260	313	368	500
KELLY BROOK					
Downstream limit of detailed study	4.9	285	405	495	735
LAMPREY RIVER					
At MacCallen Dam**	212	4,320	7,320	8,920	13,600
At USGS Gage No. 01073500	185	4,720	7,990	9,740	14,900
LITTLE COHAS BROOK					
At Industrial Road	6.70	190	365	480	770
At Harvey Road	6.30	150	310	385	540
At Litchfield Road	1.00	70	135	170	275
LITTLE RIVER NO. 1					
At the confluence with the Exeter River	13.9	345	528	624	874
LITTLE RIVER NO. 2					
At Ocean Boulevard	4.67	118	189	226	330

[1]Reach Discharge
*Data not available
**Due to diversion to Oyster River

TABLE 4 - SUMMARY OF DISCHARGES – continued

Flooding Source and Location	Drainage Area (sq. miles)	Peak Discharges (cfs)			
		10% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
LITTLE RIVER NO. 3					
Downstream limit of detailed study near Atkinson Depot Road	20.8	660	1,065	1,275	1,865
Upstream of Bryant Brook	17.1	560	900	1,075	1,585
Upstream of Seaver Brook	12.2	415	665	795	1,175
Upstream of Kelly Brook	7.0	255	405	485	715
At Plaistow-Kingston corporate limits	4.2	175	280	335	495
NESENKEAG BROOK					
At Londonderry-Litchfield corporate limits	6.90	380	720	870	1,390
At confluence with Tributary H to Nesenkeag Brook	4.80	260	500	625	1,000
PICKERING BROOK					
At Portsmouth Avenue (State Route 151)	2.45	39	48	53	62
At access road	0.80	*	*	86.54	*
PISCASSIC RIVER					
At Ice Pond	13.8	312	480	560	760
At Cuba Road	9.0	206	318	371	503
POLICY BROOK					
At Rockingham Park Inlet	5.9	350	550	660	880
At State Route 28	5.2	250	390	460	620
At a point approximately 2,000 feet above State Route 28	5.0	180	290	330	440
At a point approximately 700 feet below Main Street	4.8	100	190	210	260
UNNAMED BROOK					
At the State Route 97 bridge	0.7	70	100	120	170
PORCUPINE BROOK					
At Interstate Route 93	3.1	*	*	650	*
At Old Causeway	2.2	*	*	450	*

*Data not available

TABLE 4 - SUMMARY OF DISCHARGES – continued

Flooding Source and Location	Drainage Area (sq. miles)	Peak Discharges (cfs)			
		10% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
PORCUPINE BROOK TRIBUTARY					
At Quill Lane	0.8	*	*	210	*
POWWOW RIVER					
At Lake Gardiner Dam in Amesbury, Massachusetts	49.1	*	*	1,720	*
Downstream reach at corporate limits near Lake Gardiner	48.3	*	*	1,700	*
At Tuxbury Pond Dam in Amesbury, Massachusetts	45.9	*	*	1,640	*
Upstream reach at corporate limits in Tuxbury Pond	41.4	*	*	1,540	*
SHIELDS BROOK					
From Hornes Pond to first crossing (looking upstream) of Derry-Londonderry corporate limits[1]	6.7[2]	260	313	368	500
At first Londonderry-Derry corporate limits (looking upstream)	5.2	190	465	575	1,000
From first crossing (looking upstream) of Derry-Londonderry corporate limits to second crossing (looking upstream) of Derry-Londonderry corporate limits	5.2[2]	146	234	276	362
At confluence of Upper Beaver Brook	4.6	160	405	500	880
At second Londonderry-Derry corporate limits (looking upstream)	2.2	75	200	250	450
From second crossing (looking upstream) of Derry-Londonderry corporate limits to upstream study limit[1]	2.22	84	127	146	200

[1]Reach Discharge

[2]Drainage area at downstream limit of reach

*Data not available

TABLE 4 - SUMMARY OF DISCHARGES – continued

Flooding Source and Location	Drainage Area (sq. miles)	Peak Discharges (cfs)			
		1% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
SHOP POND					
At outlet	2.52	*	*	150	*
SPICKET RIVER					
At Hampshire Road	61.6	900	1,600	1,900	2,900
At Town Farm Road	47.9	800	1,300	1,600	2,400
At the confluence of Providence Hill Brook	40.0	700	1,200	1,400	2,100
At Arlington Mill Reservoir	26.8	350	650	750	1,100
TAYLOR BROOK					
At Island Pond	5.3	75	365	525	1,345
At outlet to Ballard Pond	4.6	10	200[1]	320[1]	960[1]
At inlet to Ballard Pond	3.4	320	820	1,005	2,000
At confluence with Tributary J to Beaver Brook	2.5	210	560	690	1,400
THE POWWOW POND SYSTEM					
At Powwow Pond/Powwow River outlet	29.6	*	*	850	*
At Country Pond outlet	14.2	*	*	410	*
At Great Pond outlet	9.96	*	*	290	*
TRIBUTARY C TO BEAVER BROOK					
At mouth	2.8	185	365	450	740
At Chester Road	2.3	120	235	310	490
TRIBUTARY D					
At Londonderry-Derry corporate limits	1.5	70	200	245	520
TRIBUTARY E TO BEAVER LAKE					
At mouth	2.8	190	350	435	700
At Chester Road	1.6	125	235	290	470

[1]Discharges reduced due to Ballard Pond Storage

*Data not available

TABLE 4 - SUMMARY OF DISCHARGES – continued

Flooding Source and Location	Drainage Area (sq. miles)	Peak Discharges (cfs)			
		10% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
TRIBUTARY E TO LITTLE COHAS BROOK					
At Beaver Lake	1.4	110	310	385	820
At Tsienneto Road	1.3	105	295	365	760
TRIBUTARY F TO BEAVER LAKE					
At Beaver Lake	7.2	250	590	725	1,350
At outlet to Adams Pond	6.0	195	475	585	1,150
TRIBUTARY G TO BEAVER BROOK					
At confluence with Beaver Brook	3.6	245	625	770	1,500
Downstream of confluence with West Running Brook	3.5	210	540	660	1,290
Upstream of confluence with West Running Brook	2.1	180	495	610	1,250
At Windham Road	1.3	120	335	410	900
TRIBUTARY H TO DREW LAKE					
At mouth	2.5	155	310	390	640
TRIBUTARY H TO NESENKEAG BROOK					
At confluence with Drew Brook	1.4	110	305	375	795
Approximately 1,000 feet upstream of Hampstead Road	1.0	25	40	120	150
TRIBUTARY J TO BLACK BROOK					
At mouth	1.6	110	140	180	285
TRIBUTARY O TO BEAVER BROOK					
At confluence with Beaver Brook	1.7	75	205	255	535
At Derry-Londonderry corporate limits	1.5	70	200	245	520

TABLE 4 - SUMMARY OF DISCHARGES – continued

Flooding Source and Location	Drainage Area (sq. miles)	Peak Discharges (cfs)			
		10% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
UPPER BEAVER BROOK					
At mouth	2.0	65	160	215	430
WASH POND					
At outlet	2.42	*	*	150	*
WASH POND TRIBUTARY					
At confluence with Wash Pond	1.03	*	*	62	*
At Kent Farm Road	0.9	*	*	54	*
WEST CHANNEL POLICY BROOK					
At Pleasant Street	2.8	*	*	200	*
At Pelham Road	2.5	*	*	380	*
WINNICUT RIVER					
At the downstream corporate limits of town of North Hampton	5.97	113	168	198	275

* Data not available

The stillwater elevations for the 1 percent annual chance flood have been determined for all detailed studied ponds and tidal areas and are summarized in Table 5, "Summary of Stillwater Elevations." For a description of the methodologies used to compute these elevations, please refer to Section 3.2, Riverine Hydraulic Analyses, in this text.

TABLE 5 – SUMMARY OF STILLWATER ELEVATIONS

Flooding Source and Location	Elevation (feet NGVD[1], NAVD[2])			
	10% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
ADAMS POND				
At Derry	326.0[1]	327.1[1]	327.3[1]	328.1[1]
ATLANTIC OCEAN				
Entire shoreline from New Castle to Seabrook	7.24[2]	7.98[2]	8.36[2]	9.43[2]
Isles of Shoals, entire shoreline	7.24[2]	7.98[2]	8.36[2]	9.43[2]
BEAVER LAKE				
At Derry	287.9[1]	289.3[1]	289.6[1]	294.0[1]
COUNTRY POND				
Entire shoreline within Kingston	*	*	120.8[1]	*
GREAT BAY				
Entire shoreline of the Squamscott River within the Exeter corporate limits to a point approximately 370 feet downstream of Chestnut Hill Avenue	6.4[2]	6.9[2]	7.2[2]	7.7[2]
Entire shoreline within Greenland and Newington, and the entire shoreline of Great Bay and Lamprey River downstream of MacCallen Dam in Newmarket	5.7[2]	6.3[2]	6.5[2]	7.1[2]
Entire shoreline of the Squamscott River within Newfields, and the entire shoreline with Stratham	6.2[2]	6.8[2]	7.0[2]	7.5[2]
GREAT POND				
Entire shoreline within Kingston	*	*	121.8[1]	*
ISLAND POND				
At the Towns of Derry and Atkinson's corporate limits, in Derry, and the entire shoreline within Hampstead	205.5[1]	206.4[1]	206.8[1]	208.2[1]
LOWER BALLARD POND				
At Derry	251.5[1]	253.6[1]	254.6[1]	256.2[1]
LOWER BEAVER LAKE				
At Derry	287.9[1]	288.9[1]	289.2[1]	290.0[1]

[1] National Geodetic Vertical Datum of 1929
[2] North American Vertical Datum of 1988
*Data not available

TABLE 5 - SUMMARY OF STILLWATER ELEVATIONS - continued

Flooding Source and Location	Elevation (feet NGVD[1], NAVD[2])			
	10% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
PISCATAQUA RIVER				
At Newington	*	*	8.3[2]	*
POWWOW POND/POWWOW RIVER				
Upstream of New Boston Road	*	*	120.8[1]	*
Upstream of Boston & Maine Railroad bridge	*	*	119.1[1]	*
Downstream of Boston & Maine Railroad bridge	*	*	118.2[1]	*
SEAVEY POND				
At Windham	*	*	248.6[1]	*
SHOP POND				
Entire shoreline within Hampstead	*	*	232.4[1]	*
SQUAMSCOTT RIVER				
Entire length within Stratham	6.2[2]	6.8[2]	7.0[2]	7.5[2]
TUXBURY POND				
Entire shoreline	*	*	100.2[1]	*
UPPER BALLARD POND				
At Derry	253.7[1]	255.5[1]	258.4[1]	259.2[1]
WASH POND				
Entire shoreline within Hampstead	*	*	234.8[1]	*
WORLD END BROOK AND POND				
At Lawrence Road in Salem	*	*	117.0[1]	*

[1] National Geodetic Vertical Datum of 1929

[2] North American Vertical Datum of 1988

* Data not available

3.2 Riverine Hydraulic Analyses

Analyses of the hydraulic characteristics of flooding from the source studied were carried out to provide estimates of the elevations of floods of the selected recurrence intervals. Users should be aware that flood elevations shown on the FIRM represent rounded whole-foot elevations and may not exactly reflect the elevations shown on the Flood Profiles or in the Floodway Data tables in the FIS report. For construction and/or floodplain management purposes, users are encouraged to use the flood elevation data presented in this FIS in conjunction with the data shown on the FIRM.

Locations of selected cross sections used in the hydraulic analyses are shown on the Flood Profiles (Exhibit 1). For stream segments for which a floodway was computed (Section 4.2), selected cross section locations are also shown on the FIRM (Exhibit 2).

On detailed study streams, all bridges, dams, and culverts were field surveyed to obtain elevation data and structural geometry.

Flood profiles were drawn showing the computed water-surface elevations for floods of the selected recurrence intervals.

The hydraulic analyses for this FIS were based on unobstructed flow. The flood elevations shown on the profiles are thus considered valid only if hydraulic structures remain unobstructed, operate properly, and do not fail.

For each community within Rockingham County that has a previously printed FIS report, the hydraulic analyses described in those reports have been compiled and are summarized below.

Precountywide Analyses

Cross sections and geometry of hydraulic structures were obtained from field surveys conducted during the 1990 field season by the study contractor. Cross-section extensions were based on information contained on USGS topographic maps (U.S. Department of the Interior, 1985, et cetera; U.S. Department of the Interior, 1981
).
For the Town of Raymond FIS report dated April 15, 1992, cross sections for the Exeter and Lamprey Rivers were obtained from field surveys and interpolation from USGS topographic maps (U.S. Department of the Interior, September 1981). Elevation data and structural geometry for bridges and culverts on both rivers were obtained from a combination of record drawings and field survey. The Prescott Road bridge at the downstream end of the Lamprey River in the Town of Raymond was under construction at the time the revised hydraulic analyses were performed. For this reason, drawings issued for construction were used to obtain hydraulic data for this bridge.

The portions of the cross sections within the limits of the channel were obtained by field survey by Kenneth A. LeClair Associates (Kenneth A. LeClair Associates, 1978). Overbank cross-sectional data were read from topographic maps at a scale of 1:2,400 (State of New Hampshire, 1970). Bridge plans were utilized to obtain elevation data and structural geometry for bridges over the streams studied in detail. Where plans were unavailable or out-of-date, bridges were also surveyed.

Cross sections for the backwater analyses of the detailed study streams were located at close intervals above and below bridges in order to compute the significant backwater effects of these structures in the developed areas. In long reaches between structures, appropriate valley cross sections were also surveyed.

For Hog Hill Brook, cross sections and geometry of hydraulic structures were obtained from field surveys conducted during the 1988 field season by the USGS. Cross-section extensions and basin characteristics were based on information contained on USGS topographic maps at a scale of 1:25,000 and 1:24,000 with contour intervals of 3 meters and 10 feet (U.S. Department of the Interior, 1985, et cetera). For Island Pond and Bryant Brook, cross sections for the backwater analyses were located at close intervals above and below bridges in order to compute the significant backwater effects of these structures in developed areas. In long reaches between structures, appropriate valley cross sections were also surveyed.

Cross-section data for the Spicket River were taken from a USACE floodplain report (U.S. Army Corps of Engineers, 1975). For Policy Brook and Unnamed Brook, cross-section data were obtained by field survey.

For the Powwow Pond/Powwow River, cross sections and elevations and structural geometry of hydraulic structures were obtained from field surveys conducted by the study contractor during the 1987 field season. Upper-end extensions of cross sections and storage areas were based on information contained on USGS topographic maps (U.S. Department of the Interior, 1981).

Water-surface elevations of floods of the selected recurrence intervals were computed using the WSPRO step-backwater computer program (Federal Highway Administration, 1990; U.S. Department of the Interior, 1989).

Water-surface elevations of floods of the selected recurrence intervals for Beaver Brook, Exeter River, Little River No. 1, Shields Brook, Homes Brook, Taylor Brook, Drew Brook, Cunningham Brook, Tributary 0 to Beaver Brook, Tributary E to Beaver Lake, Tributary F to Beaver Lake, Tributary G to Beaver Brook, and Tributary H to Nesenkeag Brook were developed using the USACE HEC-2 step- backwater computer program (U.S. Army Corps of Engineers, 1973; U.S. Army Corps of Engineers, 1977). Elevation data and structural geometry for bridges and culverts on both rivers were obtained from a combination of record drawings and field survey. The Prescott Road bridge at the downstream end of the Lamprey River in the Town of Raymond was under construction at the time the revised hydraulic analyses were performed. For this reason, drawings issued for construction were used to obtain hydraulic data for this bridge. Water-surface elevations for Spicket River of floods of the selected recurrence intervals were computed using the USACE HEC-2 step-backwater computer program (U.S. Army Corps of Engineers, 1976).

Water-surface elevations of floods of the selected recurrence intervals were computed for all detailed study streams in the community through use of the USACE HEC-2 step-backwater computer program (U.S. Army Corps of Engineers, 1977).

Water-surface elevations of floods of the selected recurrence intervals for Hog Hill

Brook, Pickering Brook, the Lamprey River, Piscassic River, West Channel Policy Brook, Porcupine Brook, and portions of the Exeter River in Fremont were computed using the SCS WSP-2 step-backwater computer program (U.S. Department of Agriculture, 1979; U.S. Department of Agriculture, 1976; U.S. Department of Agriculture, 1993).

The 1 percent annual chance flood elevations for Hog Hill Brook were computed by applying WSPRO step-backwater computer model (Federal Highway Administration, 1986; Federal Highway Administration, 1990). Starting water-surface elevations for the 1 percent annual chance flood discharge on Hog Hill at the downstream side of Haverhill Road bridge at the Salem-Atkinson corporate limits were determined using the slope/area method (Federal Highway Administration, 1986; Federal Highway Administration, 1990). Starting water-surface elevations for Bryant Brook were determined by the slope/area method. Flood profiles were drawn showing computed water-surface elevations for floods of the selected recurrence intervals.

Starting water-surface elevations for Hog Hill Brook were based on computations of elevation versus discharge at Wadleigh Falls in the Town of Lee.

Starting water-surface elevations for the Lamprey River were taken from the lower reaches of the river in the FIS report dated May 2, 1995 (FEMA, 1995). Flood profiles were drawn showing computed water-surface elevations for floods of the selected recurrence intervals.

The starting water-surface elevation for the downstream reach of the Powwow River was determined by rating the dam at the outlet of Lake Gardiner in Amesbury, Massachusetts using the weir equations referenced above. The starting water- surface elevation for Grassy Brook was computed by a slope conveyance calculation (Federal Highway Administration, 1986; U.S. Department of the Interior, 1989). The stream slope was determined from field surveys.

Starting water-surface elevations for the Exeter River in the Town of Raymond, Winnicut River, Little River No. 3, Kelly Brook, and Bryant Brook were determined by the slope/area method. Water-surface elevations of floods of the selected recurrence intervals were computed for the Little River, Kelly Branch, and Bryant Brook in the study area through use of the USACE HEC-2 step-backwater computer program (U.S. Army Corps of Engineers, 1976).

Starting water-surface elevations for the Exeter River in the Town of Exeter and Little River No. 2 were determined using critical depth. Starting water-surface elevations for the Exeter River in the Town of Fremont were based on computations of elevation versus discharge at Phillips Dam and for the Exeter River in the Town of Brentwood, starting water-surface elevations were taken from a previously studied downstream portion of the river (FEMA, October 15, 1980, FIS report; and April15, 1981, FIRM).

Starting water-surface elevations for the Little River No. 1 were determined using normal pool elevation for the Exeter River in the Town of Exeter for the 10 percent annual chance flood and the slope/area method for the 2-, 1-, and 0.2-percent annual chance floods.

Starting water-surface elevations for the 1 percent annual chance flood discharges on Hill Brook at the downstream side of the State Route 111 bridge and Shop Pond Outlet at the downstream side of Mills Shore Drive were computed using the slope-conveyance method (Federal Highway Administration, 1986 and 1990). The starting water- surface elevation for the 1 percent annual chance flood discharge on Wash Pond Tributary was the 1 percent annual chance flood elevation for Wash Pond.

For Golden Brook and Hidden Valley Brook, starting water-surface elevations were determined through normal depth analysis. For Flatrock Brook, the starting water-surface elevation was determined from a rating curve developed at the outlet of Shadow Lake.

Starting water-surface elevations for Beaver Brook were obtained from the Londonderry FIS and Hudson FIS (U.S. Department of Housing and Urban Development, 1978); Shields Brook and Tributary D from the Derry FIS (U.S. Department of Housing and Urban Development, unpublished); and Nesenkeag Brook from the Litchfield FIS (U.S. Department of Housing and Urban Development, 1977). For Black Brook, Tributary E to Beaver Lake, Tributary J to Black Brook, Tributary C to Beaver Brook, Upper Beaver Brook, Cohas Brook, Tributary H to Drew Brook, Dudley Brook, Island Pond, and Shields Brook studied by detailed methods, starting water-surface elevations were determined by normal-depth analyses.

Starting water-surface elevations for Tributary E to Little Cohas Brook and Tributary F to Beaver Lake were obtained from the Beaver Lake flood elevations, and starting water-surface elevations for Drew Brook and Taylor Brook were obtained from Island Pond flood elevations. Starting water-surface elevations for Tributary H to Nesenkeag Brook were obtained from the Drew Brook flood profile because these streams have concurrent flood peaks.

Starting water-surface elevations for the Spicket River at the dam at Arlinpon Mills Reservoir were determined from the standard Weir Formula $Q=CLH^3$. At the southern corporate limit, the 1 percent annual chance flood elevation was taken from the USACE floodplain report (U.S. Army Corps of Engineers, 1975). The starting water-surface elevation for the 10-, 2-, and 0.2-percent annual chance floods exceeded the capacity of the 60-inch culvert, and it was assumed that the water level of 124 feet (also top of the culvert) would be the ponding level for all frequency events.

Starting water-surface elevations for West Channel Policy Brook and Porcupine Brook were taken from the 1978 FIS for the Town of Salem, and a Master Drainage Study done by Weston & Sampson Engineers, Inc., respectively (U.S. Department of Housing and Urban Development, Federal Insurance Administration, 1978;

Weston and Sampson Engineers, Inc., 1988). A rating curve for World End Pond was computed by backwater analysis of flows through the Lawrence Road-Farm Road culverts.

The starting water-surface elevations for the Piscassic River were determined by computing critical depths at the Piscassic Ice Pond Dam.

Pickering Brook was studied by detailed methods in the Town of Greenland FIS, dated May 17, 1989, from a point 2,400 feet upstream of its confluence with Great Bay extending up to the corporate limits for the Town of Greenland. Starting water-surface elevations for Pickering Brook were determined by assuming critical depth at the upstream normal high tide limits of Great Bay. Water-surface elevations of floods of the selected recurrence intervals were computed through the use of the SCS WSP2 step-backwater computer program. Pickering Brook was also studied by detailed methods using the HEC-RAS hydraulic model by a LOMR effective October 6, 1999, in the Town of Portsmouth, New Hampshire, from a point approximately 2,482 feet upstream of the corporate limits for the City of Portsmouth to a point approximately 2,733 feet upstream of the corporate limits. The hydraulic analysis for Pickering Brook was extended downstream of the LOMR effective October 1999, using the HEC-RAS hydraulic model, to the corporate limits of the City of Portsmouth. The starting water-surface elevations were set at the 1 percent annual chance flood water-surface elevation at the corporate limits for the Town of Greenland.

Elevations of MacCallen Dam and the State Route 108 bridge in Newmarket were obtained from field surveys conducted by the study contractor. The 1 percent annual chance flood elevations for the Lamprey River upstream from MacCallen Dam were based upon high-water elevation data available for the April 1987 flood and data available from the FIS for the Town of Durham (FEMA, 1991).

The 1 percent annual chance flood elevation for Tuxbury Pond was determined by rating the dam at the outlet of the pond. The rating curve for the dam was determined by applying the appropriate flow over weir equations documented in a USGS publication (U.S. Department of the Interior, 1967). This elevation was also used as the starting water-surface elevation for the upstream reach of the Powwow River.

The valley portions of the cross-section data for all detailed study streams were obtained photogrammetrically by James W. Sewall Company (James W. Sewall Company, 1977); the below-water portions were obtained by field measurement by Thomas F. Moran, Inc. (Thomas F. Moran, Inc., 1977). Bridge plans were utilized to obtain elevation data and structural geometry. All bridges for which plans were unavailable or out of date were surveyed.

In those areas where the analysis indicated supercritical flow conditions, critical depth was assumed for the flood elevation because of the inherent instability of supercritical flow.

Approximate methodologies for Hidden Valley Brook include hydrologic and hydraulic calculations based on the detailed study and field investigation.

Along certain portions of Piscassic River, a profile base line is shown on the maps to represent channel distances as indicated on the flood profiles and floodway data tables.

The 1 percent annual chance flood for portions of both the Spicket River and Policy Brook was approximated, using information from an SCS Flood Prone Area Map (U.S. Department of Agriculture, 1974).

The 1 percent annual chance flood on several smaller streams was approximated using the FHBM for the Town of Salem as a guide (U.S. Department of Housing and Urban Development, 1977).

The 1 percent annual chance flood elevation for Powwow Pond/Powwow River downstream from the Boston and Maine Railroad bridge was determined by rating the dam (Trickling Falls Dam) at the outlet of the pond. For the purposes of this analysis, it was assumed that a total of 1 foot of stop logs in the gates of the dam have been removed, a practice commonly used by the Water Division of the New Hampshire Department of Environmental Services. The rating curve for the dam was determined by applying appropriate flow over weir equations documented in a USGS publication (U.S. Department of the Interior, 1967).

The 1 percent annual chance flood elevation for Powwow Pond/Powwow River upstream from the Boston and Maine Railroad bridge is controlled by the dam at the outlet of the pond and the constriction caused by the bridge opening. The flood elevation was determined by treating the opening as a culvert and passing the 1 percent annual chance flood discharge through it by applying appropriate formulas contained in a USGS publication (U.S. Department of the Interior, 1968).

The 1 percent annual chance flood elevation for Powwow Pond/Powwow River upstream from New Boston Road is influenced by the constriction caused by the twin culverts at the crossing. The flood elevation was determined by passing the 1 percent annual chance flood discharge through the twin culverts by applying appropriate formulas contained in a USGS publication (U.S. Department of the Interior, 1968). Road overflow at the site was computed by applying a step-backwater computer model (Federal Highway Administration, 1986).

The 1 percent annual chance flood elevation for Country Pond is the same as determined for Powwow Pond/Powwow River upstream from New Boston Road. Backwater from the culverts at New Boston Road extends into Country Pond. The bridge at the outlet of Country Pond does not constrict the flow sufficiently to increase elevations in the pond. To verify this fact, a step-backwater run was made through the reach (Federal Highway Administration, 1986).

The 1 percent annual chance flood elevation for Great Pond is influenced by backwater caused by the culvert under State Route 125 and Main Street bridge just downstream from the outlet. The dam at the outlet of the lake has only a small head and is drowned out during floods. Elevations upstream from State Route 125 were determined by passing the 1 percent annual chance flood discharge through the culvert by applying appropriate formulas contained in a USGS publication (U.S. Department of the Interior, 1968). The elevation upstream from State Route 125 and the 1 percent annual chance flood discharge were routed through the bridge opening of the State Route 111 crossing and into the pond using a step-backwater model (Federal Highway Administration, 1986).

Roughness factors (Manning's "n") used in the hydraulic computations were chosen by engineering judgment and were based on field observations of the streams and floodplain areas. Roughness factors for all streams studied by detailed methods are shown in Table 6, "Manning's "n" Values."

2005 Countywide Analyses

No hydraulic analyses were conducted for the 2005 countywide study.

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The Lamprey River was studied by detailed methods in the town of Newmarket from the MacCallen Dam to the upstream corporate limit for the Town of Newmarket, NH.

For the Town of Newmarket, the Lamprey River channel and structural cross section data (elevation, northing and easting) were obtained from USGS field surveys and Wright-Pierce, Inc. field surveys. The overbank portion of the cross section data for the Lamprey River was derived from the 2011 coastal LiDAR dataset described above.

Cross sections for the backwater analyses of the detailed study streams were located at close intervals above and below bridges in order to compute the significant backwater effects of these structures in the developed areas. In long reaches between structures, appropriate valley cross sections were also obtained from within channel surveys and from LiDAR on the overbanks.

Water-surface elevations of floods of the selected recurrence intervals were computed for the detailed study streams using U.S. Army Corps of Engineers HEC-RAS (version 4.1.0) step-backwater computer program (U.S. Army Corps of Engineers, January 2010). In those areas where the analysis indicated supercritical flow conditions, critical depth was assumed for the flood elevation because of the inherent instability of supercritical flow.

Starting water-surfaces for the Lamprey River were determined through computation of critical depth at the MacCallen Dam in Newmarket.

The Lamprey River HEC-RAS flood model was calibrated to the USGS streamgage 01073500 data and to the peak high-water mark data collected by the USGS along the Lamprey River after the April 2007 flood.

As in the pre-countywide analyses, roughness factors (Manning's "n") used in the coastal study hydraulic computations were chosen by engineering judgment and were based on field observations of the streams and floodplain areas. Roughness factors for the Lamprey River are also shown in Table 6, "Manning's "n" Values".

TABLE 6 – MANNING'S "n" VALUES

Stream	Channel "n"	Overbank "n"
Beaver Brook	0.020-0.055	0.040-0.100
Black Brook	0.020-0.055	0.040-0.100
Bryant Brook	0.035-0.040	0.060-0.090
Cohas Brook	0.020-0.055	0.040-0.100
Cunningham Brook	0.035-0.055	0.065-1.000
Drew Brook	0.035-0.055	0.065-1.000
Dudley Brook	0.035-0.080	0.035-0.130
Exeter River	0.020-0.080	0.020-0.150
Flatrock Brook	0.030-0.040	0.050-0.080
Golden Brook	0.022-0.045	0.060-0.080
Grassy Brook	0.030-0.040	0.140
Hidden Valley Brook	0.025-0.045	0.045-0.090
Hill Brook	0.040-0.055	0.035-0.110
Hog Hill Brook	0.035-0.065	0.075-0.100
Hornes Brook	0.035-0.055	0.065-1.000
Island Pond	0.035-0.055	0.065-1.000
Kelly Brook	0.030-0.040	0.050-0.090
Lamprey River	0.040-0.065	0.050-0.100
Little Cohas Brook	0.020-0.055	0.040-0.100
Little River No. 1	0.020-0.070	0.050-0.100
Little River No. 2	0.013-0.040	0.100
Little River No. 3	0.030-0.060	0.030-0.100
Nesenkeag Brook	0.020-0.055	0.040-0.100
Pickering Brook	0.040-0.120	0.070-0.120
Piscassic River	0.025-0.070	0.060-0.180
Policy Brook – Unnamed Brook	0.020-0.060	0.100

TABLE 6 – MANNING'S "n" VALUES - continued

Stream	Channel "n"	Overbank "n"
Porcupine Brook	0.020-0.060	0.100
Porcupine Brook Tributary	0.020-0.060	0.100
Powwow Pond System	0.025-0.035	0.030-0.090
Powwow River	0.030-0.040	0.035-0.140
Shields Brook	0.020-0.055	0.040-1.000
Spicket River	0.035	0.080
Taylor Brook (including Ballard Pond)	0.035-0.055	0.065-1.000
Tributary C to Beaver Brook	0.020-0.055	0.040-0.100
Tributary E to Beaver Lake	0.020-0.055	0.040-0.100
Tributary E to Little Cohas Brook	0.035-0.055	0.065-1.000
Tributary F to Beaver Lake	0.035-0.055	0.065-1.000
Tributary G to Beaver Brook	0.035-0.055	0.065-1.000
Tributary H to Drew Brook	0.020-0.055	0.040-0.100
Tributary H to Nesenkeag Brook	0.035-0.055	0.065-1.000
Tributary J to Black Brook	0.020-0.055	0.040-0.100
Tributary O to Beaver Brook	0.035-0.055	0.065-1.000
Upper Beaver Brook	0.020-0.055	0.040-0.100
Wash Pond Tributary	0.035-0.055	0.030-0.100
West Channel Policy Brook	0.020-0.060	0.100
Winnicut River	0.020-0.050	0.070
World End Brook and Pond	0.020-0.060	0.100

No Manning's "n" factors were assigned for computations on Catletts Creek since its flood hazard is dependent upon valley restrictions with their associated storage and not upon conveyance.

For the January 29, 2021 countywide revision, water-surface profiles for Zone A basic studies and for Zone AE detailed studies were computed through the use of the USACE HEC-RAS computer program (USACE 2010). Water surface profiles were computed for the 1-percent-annual-chance storm for the Zone A basic studies and for the 0.2, 1, 2, and 10-percent-annual chance storms for the Zone AE detailed studies.

The Zone A basic studies used the computer program Watershed Information SystEm (WISE) as a preprocessor to HEC-RAS (Watershed Concepts, 2008). WISE combined geo-referenced data from the terrain model and miscellaneous shapefiles (such as streams and cross sections). The WISE program was used to generate the input data file for HEC-RAS. Then HEC-RAS was used to determine the flood elevation at each cross section of the modeled stream. No floodway was calculated for the Zone A basic studies.

3.3 Coastal Analyses

Pre-countywide Analyses

The coastal analyses for the 2013 coastal study update supercede coastal analyses previously completed, except on the Piscataqua River, Great Bay, and the Squamscott River estuary.

Hydraulic analyses of the inland propagation of the coastal storm surge were performed for the Piscataqua River, Great Bay, and the Squamscott River estuary system using the 1-D Model. The 1-D Model is based on the hydrodynamic equations of motion and conservation of mass. The estuary system was divided into grids, with each cross section divided into areas of conveyance and storage. Cross-section data were obtained from U.S. Coast and Geodetic Survey nautical charts. The most downstream grid was located at the mouth of the Piscataqua River, while the most upstream grid was located just below the Chestnut Hill Avenue bridge over the Squamscott River in Exeter. A Chezy friction coefficient of 70 was used throughout the estuary. Wind effects were not included. Both upstream and downstream boundary conditions, the former being the function of freshwater inflow and the latter the sum of the astronomical tide and surge components, were specified initially and for the duration of the storm. Sensitivity analyses were performed for selected storm and hydraulic parameters.

2005 Countywide Analyses

No coastal analyses were conducted for the 2005 countywide study.

The January 29, 2021 Countywide Revision

The 10-, 2-, 1- and 0.2 percent annual chance stillwater elevations for the coastal areas within Rockingham County were derived from FEMA (2008) "Updating Tidal Profiles for the New England Coastline" updating the U.S. Army Corps of Engineers 1988 tidal gage profiles developed for the entire New England Coastline. The New England Tidal Flood Profiles, from Bergen Point, New York, to the Maine border with Canada, were updated by conducting new flood frequency analyses of long-term tide gage records available from the NOS and USACE. Parametric probability distributions were fit to the tide gage data using the method of L moments. The suite of probability distributions applied to the gage records included the original Pearson Type III distribution to enable comparisons between the old tidal flood profiles and the results from the new analyses. The tidal flood profiles were updated using the best fitting probability distribution, as determined by goodness-of-fit criteria.

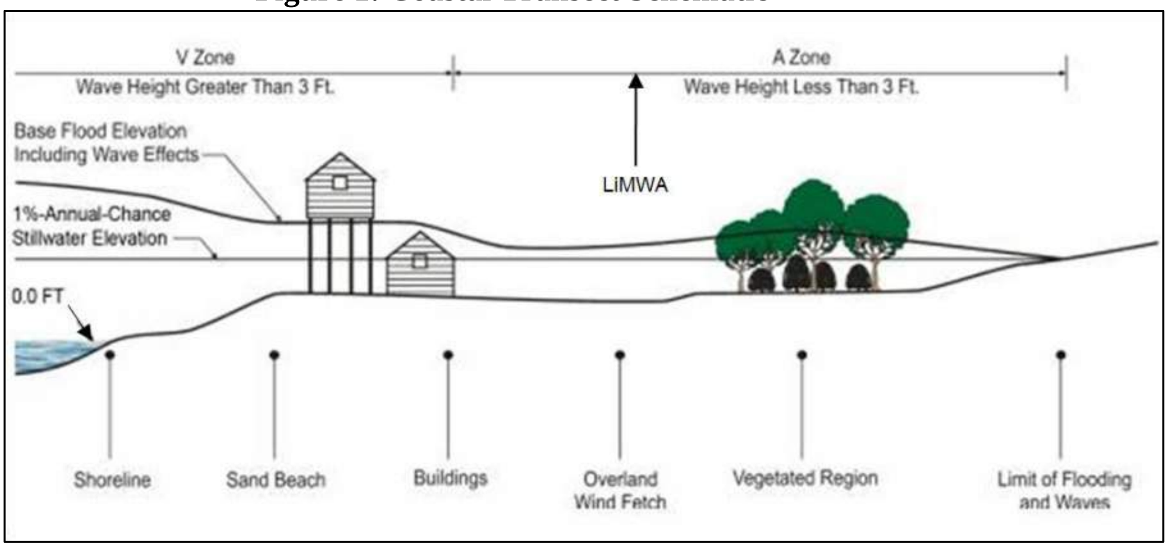
Areas of coastline subject to significant wave attack are referred to as coastal high hazard zones. The USACE has established the 3-foot breaking wave as the criterion for identifying the limit of coastal high hazard zones (U.S. Army Corps of Engineers, June 1975; U.S. Army Corps of Engineers, 1973). The 3-foot wave has been determined as the minimum size wave capable of causing major damage to conventional wood frame or brick veneer structures. Damages to structures from wave heights between 1.5

and 3 feet are similar to, but less severe than, those in areas where wave heights are greater than 3 feet. These areas have been designated as areas of moderate wave action, and areas up to the Limit of Moderate Wave Action (LiMWA) have been mapped on the FIRM.

Overland wave height analyses were performed along each transect using the FEMA Wave Hazard Analysis for Flood Insurance Studies (WHAFIS) model to determine wave heights and corresponding wave crest elevations for the areas inundated by the tidal flooding. A wave runup analysis was performed to determine the height and extent of runup beyond the limit of tidal inundation. The results of these analyses were combined into a wave envelope, which was constructed by extending the wave runup elevation seaward to its intersection with the wave crest profile.

Figure 1, "Transect Schematic," illustrates a profile for a typical transect along with the effects of energy dissipation and regeneration on a wave as it moves inland. This figure shows the wave crest elevations being decreased by obstructions, such as buildings, vegetation, and rising ground elevations, and being increased by open, unobstructed wind fetches. Figure 3 also illustrates the relationship between the local still water elevation, the ground profile and the location of the Zone V/Zone A boundary.

Figure 1: Coastal Transect Schematic



Deepwater wave characteristics used as starting wave conditions to the wave setup, overland and wave runup analyses were derived from the USACE Wave Information Studies (WIS) hindcast stations, located offshore the New Hampshire coast. The USACE website (http://wis.usace.army.mil/) provides an extreme wave analysis performed on the yearly maxima (1980-1999) at the selected stations used as the source of the 1-percent annual chance event significant wave height. The wave period associated with the 1-percent wave significant wave height was derived using a wave steepness factor of 0.035, the average wave steepness of tropical and extra-tropical events. Such wave conditions were applied to all transects facing the Atlantic Ocean shoreline. Starting wave conditions for the New Castle area, located along the Piscataqua River, were derived using a limited fetch approach within the WHAFIS model.
FEMA guidelines for Zone V mapping define H_S as the significant wave height or the

average over the highest one third of waves and T_S as the significant wave period associated with the significant wave height. Mean wave conditions are described as:

$$\overline{H} = H_s \times 0.626$$

$$\overline{T} = T_s \times 0.85$$

where $\overline{H}$ is the average wave height of all waves and $\overline{T}$ is the average wave period.

Wave heights and wave runup were computed along transects which were located perpendicular to the shoreline. The transects were located with consideration given to the physical and cultural characteristics of the land so that they would closely represent conditions in their locality. Transects were spaced close together in areas of complex topography and dense development. In areas having more uniform characteristics, the transects were spaced at larger intervals. It was also necessary to locate transects in areas where unique flooding existed and in areas where computed wave heights varied significantly between adjacent transects.

The transect profiles were obtained using topographic and bathymetric data from various sources.

The NOS Bathymetric data was acquired over several years by various agencies. The data is compiled and distributed by NOAA NOS. The bathymetric data for this project is a compilation of data acquired in 1947, 1950, 1953, 1954, 1955, 1997, 2000 and 2005. The NOS states that the accuracy of the data acquired before 1965 is difficult to determine but data acquired after 1965 must comply with standards set forth in the NOS Hydrographic Surveys Specifications and Deliverables. All bathymetric data received from the NOS has been found to meet these specifications. The data was received in Mean Low Datum and converted to NAD_1983_StatePlane_New Hampshire_FIPS_1600_Feet for use in this project.

LiDAR was collected at a 2.0 meter nominal post spacing (2.0m GSD) for approximately 8,200 mi2 of coastal areas including parts of Maine, New Hampshire, Massachusetts, Rhode Island, Connecticut, and New York, as part of the American Recovery and Reinvestment Act (ARRA) of 2010. No snow was on the ground and rivers were at or below normal levels. Some areas of the project required 1.0 meter nominal post spacing (1.0m GSD), and a required 9.25cm Vertical Accuracy. The area covered by the Piscataqua/Salmon Falls study area was covered by 1.0 meter post spacing LiDAR data and a portion of the contributing drainage area was covered by the 2.0 meter post spacing LiDAR data. A seamless Digital Elevation Model (DEM) at a 10 ft resolution was created combining the above datasets to create a base elevation for the coastal analyses.

Figures 2a and 2b, “Transect Location Map”, illustrate the location of the transects for the coastal study area.

Dune erosion was applied as per standard FEMA (2007) Guidelines and Specifications for Flood Hazard Mapping Partners methodology and VE Zones were mapped up to the extent of the Primary Frontal Dune (PFD).

Nearshore wave-induced processes, such as wave setup and wave runup, constitute a greater part of the combined wave envelope than storm surge due to location exposed to ocean waves. The Direct Integrated Method (FEMA, 2007) was used to determine wave setup along the coastline.

Wave height calculations used in this study follows the methodology described in the FEMA (2007) Guidelines and Specifications for Flood Hazard Mapping Partners. Overland wave analyses were performed along each transects using the FEMA WHAFIS 4.0 model.

Wave runup was computed in agreement with the FEMA (2005) “Procedure Memorandum No. 37” that recommends the use of the 2% wave runup for determining base flood elevations. For mild sandy beaches, Runup 2.0 was employed using mean wave conditions. Along armored shorelines, wave runup was determined using the Technical Advisory Committee for Water Retaining Structures (TAW) method (van der Meer, 2002). The Shore Protection Manual (SPM) Method was applied in cases of wave runup on vertical structures. For wave run-up at the crest of a slope that transitions to a plateau or down-slope, run-up values were determined using the “Methodology for wave run-up on a hypothetical slope” as described in the FEMA (2007) Guidelines and Specifications for Flood Hazard Mapping Partners. In areas where the wave runup overtopped the crest of a structure/bluff, the wave runup elevation was capped at 3 ft above the structure crest.

The transect data for Rockingham County is presented in Table 7, “Transect Descriptions,” which describes the location of each transect. In addition, Table 8 provides the 1-percent annual chance stillwater, wave setup and maximum wave crest elevations for each transect along the coastline.

Figure 2A: Transect Location Map – North

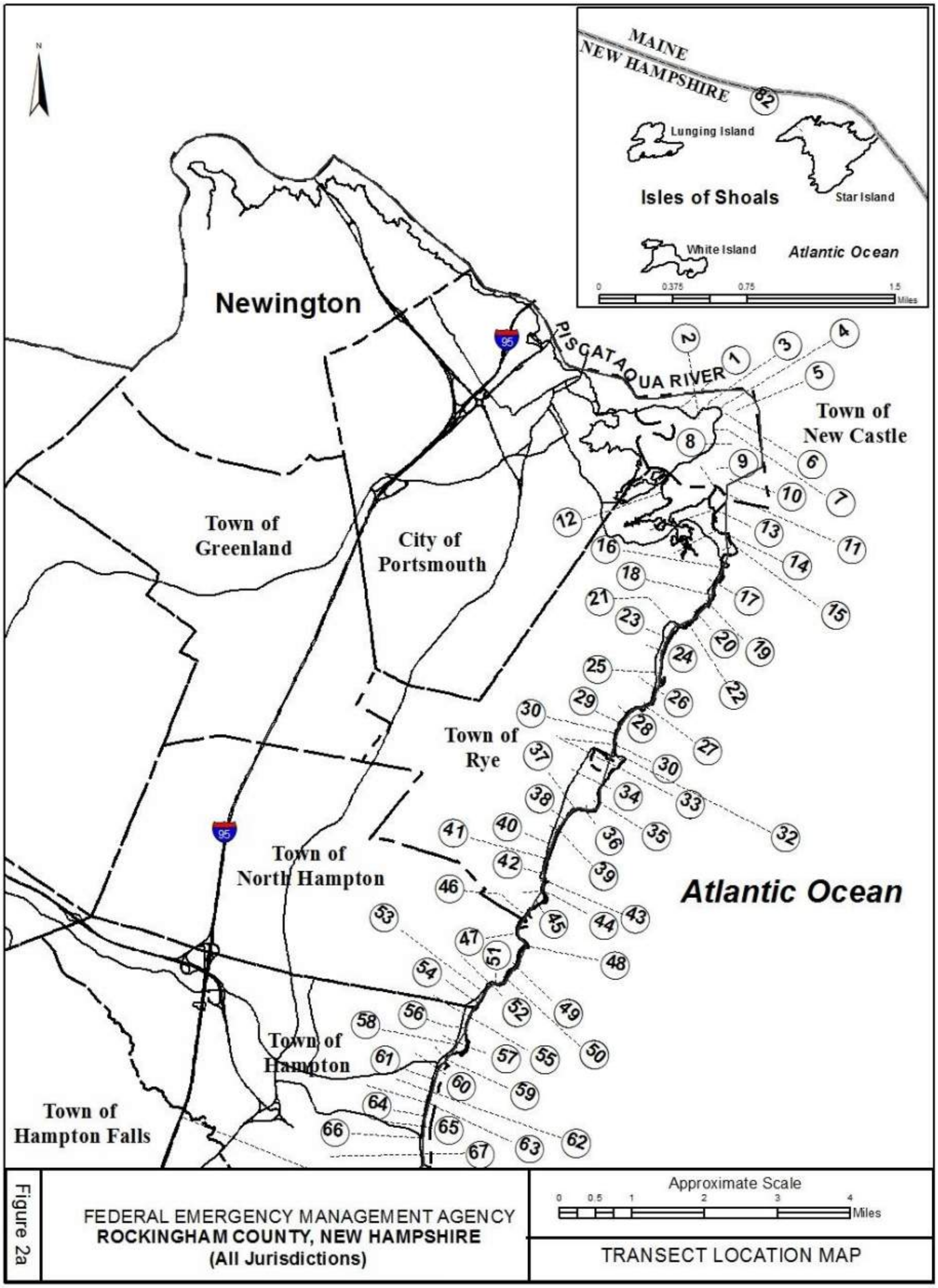


Figure 2B: Transect Location Map – South

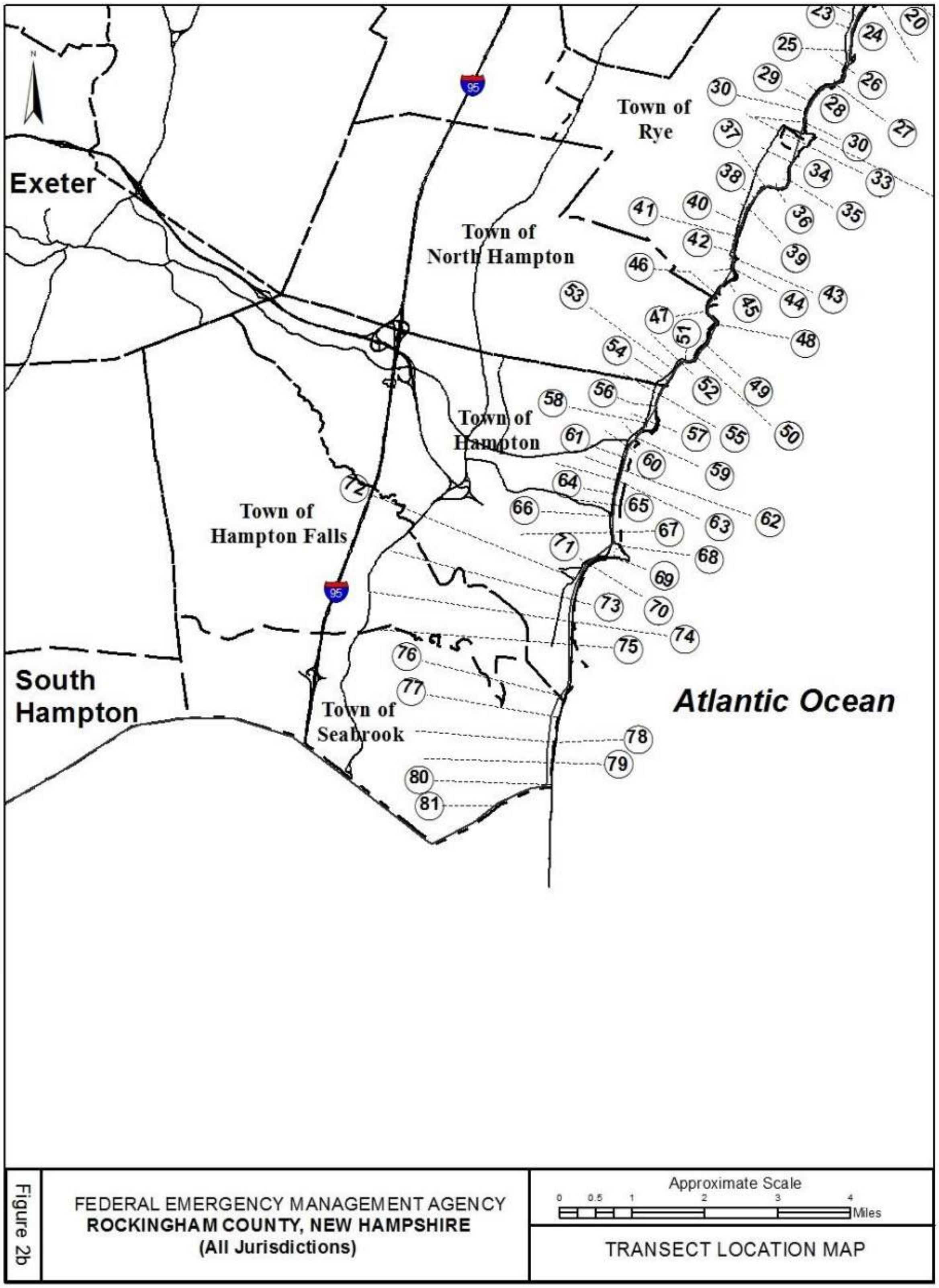


TABLE 7 – TRANSECT DESCRIPTIONS

Transect	Location	Elevation (feet NAVD88*)		
		1-Percent Annual Chance Stillwater	Wave Setup	Maximum 1-Percent Annual Chance Wave Crest[1]
1	On the Atlantic Ocean coastline, on the N side of New Castle, approximately 820 feet NE of the intersection of SR 1B (Portsmouth Ave) and Riverview Rd at N 43.0727390°, W -70.7241097°	8.36	0.66	12.37
2	On the Atlantic Ocean coastline, on the N side of New Castle, approximately 410 feet N of the intersection of SR 1B (Cranfield St) and Neals Pit Ln at N 43.071050°, W -70.718230°	8.36	0.47	11.5
3	On the Atlantic Ocean coastline, on the NE side of New Castle, approximately 100 feet E of the intersection of Elm Court and Piscataqua St at N 43.072602°, W -70.718230°	8.36	0.59	11.82
4	On the Atlantic Ocean coastline, on the NE side of New Castle, approximately 220 feet NE of the intersection of Walbach St. and Piscataqua St., at N 43.071906°, W -70.714279°	8.36	0.6	11.93
5	On the Atlantic Ocean coastline, on the NE side of New Castle, approximately 1,440 feet NE of the intersection of Wentworth Rd and Sullivan Ln, at N 43.071504°, W -70.708766°	8.36	4.29	18.5[2]
6	On the Atlantic Ocean coastline, on the NE side of New Castle, approximately 620 feet SE of the intersection of Wentworth Rd and Ocean St at N 43.069579°, W -70.712462°	8.36	3.67	18.42
7	On the Atlantic Ocean coastline, on the E side of New Castle, approximately 985 feet E of the intersection of SR 1B (Wentworth Rd) and Beach Hill Rd, at N 43.067002°, W -70.713297°.	8.36	3.63	18.36
8	On the Atlantic Ocean coastline, on the E side of New Castle, approximately 1,320 feet SE of the intersection of SR 1B (Wentworth Rd) and Tabbutt Memorial Way, at N 43.064178°, W -70.711922°.	8.36	3.95	20.1[2]
9	On the Atlantic Ocean coastline, on the SE side of New Castle, approximately 1,950 feet SE of the intersection of SR 1B (Wentworth Rd) and Wild Rose Ln, at N 43.059529°, W -70.713204°.	8.36	3.91	18.79

*North American Vertical Datum of 1988
[1]Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.
[2]Wave runup elevation

TABLE 7 – TRANSECT DESCRIPTIONS - continued

Transect	Location	Elevation (feet NAVD88*)		
		1-Percent Annual Chance Stillwater	Wave Setup	Maximum 1-Percent Annual Chance Wave Crest[1]
10	On the Atlantic Ocean coastline, on the SE side of New Castle, approximately 2,960 feet SE of the intersection of SR 1B (Wentworth Rd) and Wild Rose Ln, at N 43.056860°, W -70.711490°	8.36	2.91	17.27
11	On the Atlantic Ocean coastline, on the NE tip of Odiorne Point State Park, approximately 3,850 feet NE of the SR 1A bridge (Ocean Blvd at Pioneer Rd), at N 43.05517°, W -70.716776°	8.36	2.84	17.16
12	On the Atlantic Ocean coastline, approximately 755 feet SE of the intersection of SR 1B (Wentworth Rd) and Heather Rd, at N 43.054768°, W -70.731232°	8.36	2.84	17.16
13	On the Atlantic Ocean coastline, on the E coast of Odiorne Point State Park, approximately 2,960 feet NE of the SR 1A bridge (Ocean Blvd and Pioneer Rd), at N 43.051140°, W -70.717197°	8.36	2.65	16.88
14	On the Atlantic Ocean coastline, on the E coast of Odiorne Point State Park, approximately 1,700 feet SE of the intersection of the Odiorne Point State Park entrance and SR 1A (Ocean Blvd), at N 43.047073°, W -70.71641°	8.36	2.62	16.83
15	On the Atlantic Ocean coastline, on the E coast of Odiorne Point State Park, approximately 3,200 feet SE of the intersection of the Odiorne Point State Park entrance and SR 1A (Ocean Blvd), at N 43.0438622°, W -70.711755°	8.36	3.16	17.65
16	On the Atlantic Ocean coastline, approximately 1,320 feet NE of the intersection of Pollack Dr and SR 1A (Ocean Blvd), at N 43.039461°, W -70.715128°	8.36	3.17	17.67
17	On the Atlantic Ocean coastline, approximately 208 feet NE of the intersection of Pollack Dr and SR 1A (Ocean Blvd), at N 43.036399°, W -70.717116°	8.36	3.25	17.79
18	On the Atlantic Ocean coastline, approximately 551 feet SE of the intersection of Parsons Road and SR 1A (Ocean Blvd), at N 43.033897°, W -70.717479°	8.36	3.22	17.74

*North American Vertical Datum of 1988
[1]Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.

TABLE 7 – TRANSECT DESCRIPTIONS - continued

Transect	Location	Elevation (feet NAVD88*)		
		1-Percent Annual Chance Stillwater	Wave Setup	Maximum 1-Percent Annual Chance Wave Crest[1]
19	On the Atlantic Ocean coastline, approximately 280 feet SE of the intersection of Neptune Dr and SR 1A (Ocean Blvd), at N 43.032123°, W -70.718778°	8.36	3.40	18.10
20	On the Atlantic Ocean coastline, approximately 300 feet S of the intersection of Shoals View Dr and SR 1A (Ocean Blvd), at N 43.03039°, W -70.722316°	8.36	3.27	20.1[2]
21	On the Atlantic Ocean coastline, approximately 694 feet E of the intersection of Fairhill Ave and SR 1A (Ocean Blvd), at N 43.028312°, W -70.724441°	8.36	3.36	17.95
22	On the Atlantic Ocean coastline, approximately 680 feet SE of the intersection of Marsh Rd and SR 1A (Ocean Blvd) at Wallis Sands State Park, at N 43.02738°, W -70.727493°	8.36	3.35	17.94
23	On the Atlantic Ocean coastline, approximately 1,300 feet S of the intersection of SR 1A (Ocean Blvd) and Marsh Rd near Wallis Sands State Park, at N 43.025270°, W -70.729617°	8.36	3.28	17.83
24	On the Atlantic Ocean coastline, approximately 671 feet NE of the intersection of SR 1A (Ocean Blvd) and Wallis Rd near Wallis Sands State Park, at N 43.022747°, W -70.731182°	8.36	3.39	18.00
25	On the Atlantic Ocean coastline, approximately 1,270 feet SE of the intersection of SR 1A (Ocean Blvd) and Wallis Rd, at N 43.018597°, W -70.732173°	8.36	3.39	20.00[2]
26	On the Atlantic Ocean coastline, approximately 330 feet NE of the intersection of SR 1A (Ocean Blvd) and Highland Park Ave, at N 43.015226°, W -70.733395°	8.36	3.36	18.8[2]
27	On the Atlantic Ocean coastline, approximately 1.200 feet SW of the intersection of SR 1A (Ocean Blvd) and Highland Park Ave, at N 43.011954°, W -70.736492°	8.36	3.15	17.63
28	On the Atlantic Ocean coastline, approximately 260 feet S of the intersection of SR 1A (Ocean Blvd) and Washington Rd, at N 43.0102309°, W -70.741415°	8.36	3.21	19.2[2]

*North American Vertical Datum of 1988
[1]Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.
[2]Wave runup elevation

TABLE 7 – TRANSECT DESCRIPTIONS - continued

Transect	Location	Elevation (feet NAVD88*)		
		1-Percent Annual Chance Stillwater	Wave Setup	Maximum 1-Percent Annual Chance Wave Crest[1]
29	On the Atlantic Ocean coastline, approximately 1,015 feet SW of the intersection of SR 1A (Ocean Blvd) and Washington Rd, at N 43.0084721°, W -70.7431°	8.36	3.28	20.7[2]
30	On the Atlantic Ocean coastline, approximately 1,700 feet SW of the intersection of SR 1A (Ocean Blvd) and Washington Rd, at N 43.006570°, W -70.744378°	8.36	3.31	21.3[2]
31	On the Atlantic Ocean coastline, approximately 2,750 feet NE of the intersection of SR 1A (Ocean Blvd) and Harbor Rd, at N 43.004349°,W -70.7448644°	8.36	3.30	19.69[2]
32	On the Atlantic Ocean coastline, approximately 3,120 feet NE of the intersection of SR 1A (Ocean Blvd) and Harbor Rd near Rye Harbor State Park, at N 43.001628°, W -70.7422843°	8.36	3.38	17.98
33	On the Atlantic Ocean coastline, approximately 2,590 feet E of the intersection of SR 1A (Ocean Blvd) and Harbor Rd near Rye Harbor State Park, at N 42.999736°, W -70.744238°	8.36	3.39	18.00
34	On the Atlantic Ocean coastline, approximately 2,100 feet SE of the intersection of SR 1A (Ocean Blvd) and Harbor Rd, at N 42.996333°, W -70.748637°	8.36	3.11	18.2[2]
35	On the Atlantic Ocean coastline, approximately 1,000 feet E of the intersection of Wildwood Ln and Locke Rd, at N 42.992949°, W -70.749540°	8.36	3.14	19.4[2]
36	On the Atlantic Ocean coastline, approximately 700 ft SE of the intersection of Wildwood Ln and Locke Rd, at N 42.991261°, W -70.753217	8.36	2.63	19.4[2]
37	On the Atlantic Ocean coastline, approximately 800 feet E of the intersection of SR 1A (Ocean Blvd) and Jenness Rd, at N 42.991335°, W -70.755859°	8.36	3.15	17.63
38	On the Atlantic Ocean coastline, approximately 600 feet SE of the intersection of SR 1A (Ocean Blvd) and Cable Rd, at N 42.989358°, W -70.75873°	8.36	3.19	17.70

*North American Vertical Datum of 1988
[1]Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.
[2]Wave runup elevation

TABLE 7 – TRANSECT DESCRIPTIONS - continued

Transect	Location	Elevation (feet NAVD88*)		
		1-Percent Annual Chance Stillwater	Wave Setup	Maximum 1-Percent Annual Chance Wave Crest[1]
39	On the Atlantic Ocean coastline, approximately 460 feet SE of the intersection of SR 1A (Ocean Blvd) and Myrica Ave, at N 42.987200°, W -70.760358°	8.36	3.20	17.71
40	On the Atlantic Ocean coastline, approximately 714 feet SE of the intersection of SR 1A (Ocean Blvd) and Perkins Rd, at N 42.984288°, W -70.761968°	8.36	3.18	17.68
41	On the Atlantic Ocean coastline, approximately 1,640 feet S of the intersection of SR 1A (Ocean Blvd) and Perkins Rd, at N 42.9816514°, W -70.7634314°	8.36	3.19	20.90[2]
42	On the Atlantic Ocean coastline, approximately 432 feet NE of the intersection of SR 1A (Ocean Blvd) and Sea Rd, at N 42.978573°, W -70.764351°	8.36	2.99	17.38
43	On the Atlantic Ocean coastline, approximately 620 feet SE of the intersection of SR 1A (Ocean Blvd) and Sea Rd, at N 42.977074°, W -70.763627°	8.36	3.11	17.57
44	On the Atlantic Ocean coastline, approximately 940 feet SE of the intersection of SR 1A (Ocean Blvd) and Sea Rd, at N 42.975361°, W -70.764815°	8.36	3.17	17.90[2]
45	On the Atlantic Ocean coastline, approximately 690 feet NE of the intersection of SR 1A (Ocean Blvd) and Central Rd, at N 42.972524°, W -70.766268°	8.36	3.28	17.60[2]
46	On the Atlantic Ocean coastline, approximately 536 feet SW of the intersection of SR 1A (Ocean Blvd) and Central Rd, at N 42.970282°, W -70.769807°	8.36	3.30	20.10[2]
47	On the Atlantic Ocean coastline, approximately 505 feet NW of the intersection of SR 1A (Ocean Blvd) and Willow Ave, at N 42.966904°, W -70.772041°	8.36	2.85	23.60[2]
48	On the Atlantic Ocean coastline, approximately 784 feet SE of the intersection of SR 1A (Ocean Blvd) and Willow Ave, at N 42.964257°, W -70.769130°	8.36	3.46	21.73

*North American Vertical Datum of 1988
[1]Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.
[2]Wave runup elevation

TABLE 7 – TRANSECT DESCRIPTIONS - continued

Transect	Location	Elevation (feet NAVD88*) 1-Percent Annual Chance Stillwater	Wave Setup	Maximum 1-Percent Annual Chance Wave Crest[1]
49	On the Atlantic Ocean coastline, approximately 1,028 feet NE of the intersection of SR 1A (Ocean Blvd) and Atlantic Ave, at N 42.960135°, W -70.772513°	8.36	3.34	18.30[2]
50	On the Atlantic Ocean coastline, approximately 286 feet SE of the intersection of SR 1A (Ocean Blvd) and Atlantic Ave, at N 42.957757°, W -70.775255°	8.36	3.34	26.9[2]
51	On the Atlantic Ocean coastline, approximately 202 feet SE of the intersection of SR 1A (Ocean Blvd) and Sea Rd, at N 42.956776°, W -70.778349°	8.36	2.54	16.71
52	On the Atlantic Ocean coastline, approximately 359 feet SW of the intersection of SR 1A (Ocean Blvd) and Sea Rd, at N 42.956563°, W -70.779446°	8.36	3.34	17.92
53	On the Atlantic Ocean coastline, approximately 1,430 feet NE of the intersection of SR 1A (Ocean Blvd) and Appledore Ave, at N 42.954856°, W -70.781128°	8.36	3.34	17.92
54	On the Atlantic Ocean coastline, approximately 802 feet E of the intersection of SR 1A (Ocean Blvd) and Appledore Ave, at N 42.952824°, W -70.782864°	8.36	3.39	18.2[2]
55	On the Atlantic Ocean coastline, approximately 948 feet SE of the intersection of SR 1A (Ocean Blvd) and Appledore Ave, at N 42.950306°, W -70.785469°	8.36	3.34	18.00[2]
56	On the Atlantic Ocean coastline, approximately 850 feet SE of the intersection of SR 1A (Ocean Blvd) and Huckleberry Ln., at N 42.948053°, W -70.78646°	8.36	3.32	20.00[2]
57	On the Atlantic Ocean coastline, approximately 1,372 feet SE of the intersection of SR 1A (Ocean Blvd) and Cranberry Ln., at N 42.944272°, W -70.785747°	8.36	3.30	19.60[2]
58	On the Atlantic Ocean coastline, approximately 579 feet E of the intersection of SR 1A (Ocean Blvd) and Smith Ave, at N 42.943092°, W -70.789112°	8.36	2.54	17.86

*North American Vertical Datum of 1988
[1]Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.
[2]Wave runup elevation

TABLE 7 – TRANSECT DESCRIPTIONS - continued

Transect	Location	Elevation (feet NAVD88*)		
		1-Percent Annual Chance Stillwater	Wave Setup	Maximum 1-Percent Annual Chance Wave Crest[1]
59	On the Atlantic Ocean coastline, approximately 320 feet SE of the intersection of SR 1A (Ocean Blvd) and Cusack Rd, at N 42.941746°, W -70.791868°	8.36	3.15	16.70
60	On the Atlantic Ocean coastline, approximately 472 feet S of the intersection of SR 1A (Ocean Blvd) and High St, at N 42.939897°, W -70.7940118°	8.36	3.19	17.70
61	On the Atlantic Ocean coastline, approximately 1,262 feet SW of the intersection of SR 1A (Ocean Blvd) and High St, at N 42.937821°, W -70.7949304°	8.36	3.24	17.77
62	On the Atlantic Ocean coastline, approximately 2,160 feet SW of the intersection of SR 1A (Ocean Blvd) and High St, at N 42.935393°, W -70.796118°	8.36	3.22	17.74
63	On the Atlantic Ocean coastline, approximately 3,010 feet SW of the intersection of SR 1A (Ocean Blvd) and High St, at N 42.933136°, W -70.796850°	8.36	3.22	17.74
64	On the Atlantic Ocean coastline, approximately 1,430 feet NE of the intersection of SR 1A (Ocean Blvd) and SR 101E (Winnacunnet Rd), at N 42.930480°, W -70.797669°	8.36	3.26	17.79
65	On the Atlantic Ocean coastline, approximately 630 feet NE of the intersection of SR 1A (Ocean Blvd) and SR 101E (Winnacunnet Rd), at N 42.928423°, W -70.798082°	8.36	3.20	17.70
66	On the Atlantic Ocean coastline, approximately 254 feet SE of the intersection of SR 1A (Ocean Blvd) and SR 101E (Winnacunnet Rd), at N 42.926085°, W -70.798377°	8.36	3.19	17.70
67	On the Atlantic Ocean coastline, approximately 1,370 feet SE of the intersection of SR 1A (Ocean Blvd) and SR 101E (Winnacunnet Rd), at N 42.922896°, W -70.798485°	8.36	3.30	17.86
68	On the Atlantic Ocean coastline, approximately 681 feet SE of the intersection of SR 1A (Ocean Blvd) and Dumas Ave, at N 42.920102°, W -70.796257°	8.36	3.22	17.74

*North American Vertical Datum of 1988
[1]Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.

TABLE 7 – TRANSECT DESCRIPTIONS - continued

Transect	Location	Elevation (feet NAVD88*)		
		1-Percent Annual Chance Stillwater	Wave Setup	Maximum 1-Percent Annual Chance Wave Crest[1]
69	On the Atlantic Ocean coastline, approximately 527 feet SE of the intersection of SR 1A (Ocean Blvd) and Great Boars Head Ave, at N 42.917779°, W -70.798271°	8.36	2.42	16.53
70	On the Atlantic Ocean coastline, approximately 300 feet SW of the intersection of SR 1A (Ocean Blvd) and Anchor St, at N 42.917694°, W -70.802532°	8.36	2.75	17.03
71	On the Atlantic Ocean coastline, approximately 340 feet SE of the intersection of SR 1A (Ocean Blvd) and Tilton St, at N 42.916583°, W -70.805151°	8.36	3.14	17.62
72	On the Atlantic Ocean coastline, approximately 376 feet E of the intersection of SR 1A (Ocean Blvd) and SR 101 (Highland Ave), at N 42.913316°, W -70.807427°	8.36	3.14	17.62
73	On the Atlantic Ocean coastline, approximately 1,430 feet S of the intersection of SR 1A (Ocean Blvd) and SR 101 (Highland Ave), at N 42.909361°, W -70.809015°	8.36	3.13	17.60
74	On the Atlantic Ocean coastline, approximately 976 feet NE of the intersection of SR 1A (Ocean Blvd) and Bradford Ave., at N 42.905084°, W -70.809722°	8.36	3.13	17.60
75	On the Atlantic Ocean coastline, approximately 1,200 feet NE of the intersection of SR 1A (Ocean Blvd) and Bradford Ave, at N 42.900506°, W -70.809943°	8.36	3.13	17.6
76	On the Atlantic Ocean coastline, approximately 347 feet SE of the intersection of Ashland St and Ocean Dr, at N 42.890035°, W -70.811957°	8.36	3.28	17.83
77	On the Atlantic Ocean coastline, approximately 1,425 feet SE of the intersection of SR 1A (Ocean Blvd) and Hooksett St, at N 42.885943°, W -70.813515°	8.36	3.27	17.82
78	On the Atlantic Ocean coastline, approximately 990 feet SE of the intersection of SR 1A (Ocean Blvd) and Andover St, at N 42.880987°, W -70.814699°	8.36	3.34	17.92

*North American Vertical Datum of 1988
[1]Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.

TABLE 7 – TRANSECT DESCRIPTIONS - continued

Transect	Location	Elevation (feet NAVD88*)		
		1-Percent Annual Chance Stillwater	Wave Setup	Maximum 1-Percent Annual Chance Wave Crest[1]
79	On the Atlantic Ocean coastline, approximately 802 feet SE of the intersection of SR 1A (Ocean Blvd) and Salem St, at N 42.8769443°, W -70.815328°	8.36	3.36	17.95
80	On the Atlantic Ocean coastline, approximately 675 feet NE of the intersection of SR 1A (Ocean Blvd) and SR 286, at N 42.872535°, W -70.815788°	8.36	3.23	17.76
81	On the Atlantic Ocean coastline, approximately 1710 feet SE of the intersection of SR 1A (Ocean Blvd) and SR 286, at N 42.868108°, W -70.815855°	8.36	**	10.04
82	On the north coastline of Star Island, within the Isles of Shoals, approximately 530 feet SW from the seaward tip of the Star Island's dock at N 42.977967°, W -70.615943°	8.36	5.2	33.58[2]

*North American Vertical Datum of 1988
**Wave setup not applied to NH portion of transect, which is inland from setup impacts.
[1]Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.

In Table 8, "Transect Data," the flood hazard zone and base flood elevations for each transect are provided, along with the 10-, 2-, 1-, and 0.2-percent annual chance stillwater elevations and the 1% total water elevation (includes setup).

TABLE 8 – TRANSECT DATA

Transect	Stillwater Elevation (feet NAVD88*)				Total Water Elevation 1-Percent Annual Chance[1]	Zone	Base Flood Elevation* (feet NAVD88**)
	10-Percent Annual Chance	2-Percent Annual Chance	1-Percent Annual Chance	0.2-Percent Annual Chance			
1	7.24	7.98	8.36	9.43	9.02	VE	11-12
						AE	9-10

*Due to map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted.
**North American Vertical Datum of 1988
[1]Including stillwater elevation and effects of wave setup.

TABLE 8 – TRANSECT DATA – continued

Transect	Stillwater Elevation (feet NAVD88*)				Total Water Elevation 1-Percent Annual Chance[1]	Zone	Base Flood Elevation* (feet NAVD88**)
	10-Percent Annual Chance	2-Percent Annual Chance	1-Percent Annual Chance	0.2-Percent Annual Chance			
2	7.24	7.98	8.36	9.43	8.83	VE	11-12
						AE	9-10
3	7.24	7.98	8.36	9.43	8.95	VE	11-12
						AE	9-10
4	7.24	7.98	8.36	9.43	8.96	VE	11-12
						AE	9-10
5	7.24	7.98	8.36	9.43	12.65	VE	19[2]
						AE	19[2]
						AO	3
6	7.24	7.98	8.36	9.43	12.03	VE	14-18
						AE	12-14
7	7.24	7.98	8.36	9.43	11.99	VE	14-18
						AE	12-14
8	7.24	7.98	8.36	9.43	12.31	VE	20[2]
						AE	18[2]
9	7.24	7.98	8.36	9.43	12.27	VE	14-18
						AE	12-14
10	7.24	7.98	8.36	9.43	11.27	VE	16[2]-17
						AE	16[2]
						AO	3
11	7.24	7.98	8.36	9.43	11.2	VE	13-17
						AE	11-13
12	7.24	7.98	8.36	9.43	11.2	VE	13-17
						AE	11-13
13	7.24	7.98	8.36	9.43	11.01	VE	13-17
						AE	11-13

*Due to map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted.
**North American Vertical Datum of 1988
[1]Including stillwater elevation and effects of wave setup
[2]Wave runup elevation.

TABLE 8 – TRANSECT DATA – continued

Transect	Stillwater Elevation (feet NAVD88*) 10-Percent Annual Chance	2-Percent Annual Chance	1-Percent Annual Chance	0.2-Percent Annual Chance	Total Water Elevation 1-Percent Annual Chance[1]	Zone	Base Flood Elevation* (feet NAVD88**)
14	7.24	7.98	8.36	9.43	10.98	VE	13-17
						AE	11-13
						AE	8-10
15	7.24	7.98	8.36	9.43	11.52	VE	15[2]-18
						AE	15[2]
						AO	3
16	7.24	7.98	8.36	9.43	11.53	VE	16[2]-18
						AE	16[2]
						AO	3
						AE	8-9
17	7.24	7.98	8.36	9.43	11.61	VE	17[2]-18
						AE	17[2]
						AO	3
18	7.24	7.98	8.36	9.43	11.58	VE	17[2]-18
						AE	17[2]
						AO	3
						AE	8-9
19	7.24	7.98	8.36	9.43	11.76	VE	16[2]-18
						AE	16[2]
						AO	3
20	7.24	7.98	8.36	9.43	11.63	VE	20[2]
						AE	20[2]
						AO	3
21	7.24	7.98	8.36	9.43	11.72	VE	14-18
						AE	12-14
22	7.24	7.98	8.36	9.43	11.71	VE	14-18
						AE	8-14
23	7.24	7.98	8.36	9.43	11.64	VE	14-18
						AE	8-9

*Due to map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted.
**North American Vertical Datum of 1988
[1]Including stillwater elevation and effects of wave setup
[2]Wave runup elevation.

TABLE 8 – TRANSECT DATA – continued

Transect	Stillwater Elevation (feet NAVD88*)				Total Water Elevation 1-Percent Annual Chance[1]	Zone	Base Flood Elevation* (feet NAVD88**)
	10-Percent Annual Chance	2-Percent Annual Chance	1-Percent Annual Chance	0.2-Percent Annual Chance			
24	7.24	7.98	8.36	9.43	11.75	VE	14-18
						AE	8-9
25	7.24	7.98	8.36	9.43	11.75	VE	20[2]
						AE	20[2]
						AO	3
						AE	8-10
26	7.24	7.98	8.36	9.43	11.72	VE	19
						AE	19[2]
						AO	3
						AE	8-10
27	7.24	7.98	8.36	9.43	11.51	VE	17[2]-18
						AE	17[2]
						AO	3
28	7.24	7.98	8.36	9.43	11.57	VE	19[2]
						AE	19[2]
						AO	3
						AE	8-9
29	7.24	7.98	8.36	9.43	11.64	VE	20[2]
						AO	3
						AE	8-9
30	7.24	7.98	8.36	9.43	11.67	VE	21[2]
						AE	21[2]
						AO	3
						AE	8-10
31	7.24	7.98	8.36	9.43	11.66	VE	20[2]
						AE	20[2]
						AO	3
						AE	8-10
32	7.24	7.98	8.36	9.43	11.74	VE	16[2]-18
						AE	16[2]
						AO	3

*Due to map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted.
**North American Vertical Datum of 1988
[1]Including stillwater elevation and effects of wave setup
[2]Wave runup elevation.

TABLE 8 – TRANSECT DATA – continued

Transect	Stillwater Elevation (feet NAVD88*)				Total Water Elevation 1-Percent Annual Chance[1]	Zone	Base Flood Elevation* (feet NAVD88**)
	10-Percent Annual Chance	2-Percent Annual Chance	1-Percent Annual Chance	0.2-Percent Annual Chance			
33	7.24	7.98	8.36	9.43	11.75	VE	14-18
						AE	10-14
						AE	8-10
34	7.24	7.98	8.36	9.43	11.47	VE	18
						AO	3
						AE	8-10
35	7.24	7.98	8.36	9.43	11.50	VE	19[2]
						AE	19[2]
						AO	3
36	7.24	7.98	8.36	9.43	10.99	VE	19[2]
						AE	19[2]
						AO	3
37	7.24	7.98	8.36	9.43	11.51	VE	14-18
						AE	8-13
38	7.24	7.98	8.36	9.43	11.55	VE	14-18
39	7.24	7.98	8.36	9.43	11.56	VE	15[2]-18
						AO	3
40	7.24	7.98	8.36	9.43	11.54	VE	17[2]-18
						AE	17[2]
						AE	8-9
41	7.24	7.98	8.36	9.43	11.55	VE	21[2]
						AE	21[2]
						AO	3
						AE	8-9
42	7.24	7.98	8.36	9.43	11.35	VE	16[2]-17
						AE	16[2]
						AO	3
						AE	8-9

*Due to map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted.
**North American Vertical Datum of 1988
[1]Including stillwater elevation and effects of wave setup
[2]Wave runup elevation.

TABLE 8 – TRANSECT DATA – continued

Transect	Stillwater Elevation (feet NAVD88*)				Total Water Elevation 1-Percent Annual Chance[1]	Zone	Base Flood Elevation* (feet NAVD88**)
	10-Percent Annual Chance	2-Percent Annual Chance	1-Percent Annual Chance	0.2-Percent Annual Chance			
43	7.24	7.98	8.36	9.43	11.47	VE	16[2]-18
						AE	16[2]
						AO	3
44	7.24	7.98	8.36	9.43	11.53	VE	18[2]
						AE	18[2]
						AO	3
45	7.24	7.98	8.36	9.43	11.64	VE	18[2]
						AE	18[2]
						AO	3
46	7.24	7.98	8.36	9.43	11.66	VE	20[2]
						AE	20[2]
						AO	3
						AE	8-9
47	7.24	7.98	8.36	9.43	11.21	VE	24[2]
						AE	24[2]
						AO	3
						AE	8-9
48	7.24	7.98	8.36	9.43	11.82	VE	22[2]
						AE	22[2]
49	7.24	7.98	8.36	9.43	11.70	VE	18[2]
						AE	18[2]
50	7.24	7.98	8.36	9.43	11.70	VE	27[2]
						AE	27[2]
51	7.24	7.98	8.36	9.43	10.90	VE	16[2]-17
						AE	16[2]
						AO	3
						AE	8-9
52	7.24	7.98	8.36	9.43	11.70	VE	17[2]-18
						AE	17[2]
						AO	3
						AE	8-10

*Due to map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted.
**North American Vertical Datum of 1988
[1]Including stillwater elevation and effects of wave setup
[2]Wave runup elevation.

TABLE 8 – TRANSECT DATA – continued

Transect	Stillwater Elevation (feet NAVD88*)				Total Water Elevation 1-Percent Annual Chance[1]	Zone	Base Flood Elevation* (feet NAVD88**)
	10-Percent Annual Chance	2-Percent Annual Chance	1-Percent Annual Chance	0.2-Percent Annual Chance			
53	7.24	7.98	8.36	9.43	11.70	VE	17[2]-18
							17[2]
						AO	3
						AE	8-10
54	7.24	7.98	8.36	9.43	11.75	VE	18[2]
						AE	18[2]
						AO	3
						AE	8-10
55	7.24	7.98	8.36	9.43	11.70	VE	18[2]
						AE	18[2]
						AO	3
						AE	8-9
56	7.24	7.98	8.36	9.43	11.68	VE	20[2]
						AO	2
						AE	8
57	7.24	7.98	8.36	9.43	11.66	VE	14-18
						AE	12
						AE	8-9
58	7.24	7.98	8.36	9.43	10.90	VE	13-17
						AE	11-12
59	7.24	7.98	8.36	9.43	11.51	VE	15[2]-18
						AE	15[2]
						AO	3
						AE	8-9
60	7.24	7.98	8.36	9.43	11.55	VE	16[2]-18
						AE	16[2]
						AO	3
						AE	8-9
61	7.24	7.98	8.36	9.43	11.60	VE	16[2]-18
						AE	16[2]
						AO	3
						AE	8-10

*Due to map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted.
**North American Vertical Datum of 1988
[1]Including stillwater elevation and effects of wave setup
[2]Wave runup elevation.

TABLE 8 – TRANSECT DATA – continued

Transect	Stillwater Elevation (feet NAVD88*)				Total Water Elevation 1-Percent Annual Chance[1]	Zone	Base Flood Elevation* (feet NAVD88**)
	10-Percent Annual Chance	2-Percent Annual Chance	1-Percent Annual Chance	0.2-Percent Annual Chance			
62	7.24	7.98	8.36	9.43	11.58	VE	16[2]-18
						AE	16[2]
						AO	3
						AE	8-10
63	7.24	7.98	8.36	9.43	11.58	VE	16[2]-18
						AE	16[2]
						AO	3
						AE	8-10
64	7.24	7.98	8.36	9.43	11.62	VE	15[2]-18
						AE	15[2]
						AO	3
						AE	8-10
65	7.24	7.98	8.36	9.43	11.56	VE	15[2]-18
						AE	15[2]
						AO	3
						AE	8-9
66	7.24	7.98	8.36	9.43	11.55	VE	15[2]-18
						AE	15[2]
						AO	3
						AE	8-10
67	7.24	7.98	8.36	9.43	11.66	VE	16[2]-18
						AE	16[2]
						AO	3
						AE	8-10
68	7.24	7.98	8.36	9.43	11.58	VE	16[2]-18
						AE	16[2]
69	7.24	7.98	8.36	9.43	10.78	VE	13[2]-17
						AE	13[2]
70	7.24	7.98	8.36	9.43	11.11	VE	16[2]-17
						AE	16[2]
						AO	3
						AE	8-10

*Due to map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted.
**North American Vertical Datum of 1988
[1]Including stillwater elevation and effects of wave setup
[2]Wave runup elevation.

TABLE 8 – TRANSECT DATA – continued

Transect	Stillwater Elevation (feet NAVD88*)				Total Water Elevation 1-Percent Annual Chance[1]	Zone	Base Flood Elevation* (feet NAVD88**)
	10-Percent Annual Chance	2-Percent Annual Chance	1-Percent Annual Chance	0.2-Percent Annual Chance			
71	7.24	7.98	8.36	9.43	11.50	VE	12[2]-18
						AE	12[2]
						AE	8-10
72	7.24	7.98	8.36	9.43	11.50	VE	13[2]-18
						AE	13[2]
						AE	8-10
73	7.24	7.98	8.36	9.43	11.49	VE	13[2]-18
						AE	13[2]
						AE	8-10
74	7.24	7.98	8.36	9.43	11.49	VE	13[2]-18
						AE	13[2]
						AE	8-10
75	7.24	7.98	8.36	9.43	11.49	VE	14-18
						AE	8-10
76	7.24	7.98	8.36	9.43	11.64	VE	14-18
						AE	8-10
77	7.24	7.98	8.36	9.43	11.63	VE	14-18
						AE	8-10
78	7.24	7.98	8.36	9.43	11.70	VE	14-18
						AE	8-10
79	7.24	7.98	8.36	9.43	11.72	VE	14-18
						AE	8-10
80	7.24	7.98	8.36	9.43	11.59	VE	14-18
						AE	8-10
81	7.24	7.98	8.36	9.43	8.36	AE	8-10
82	7.24	7.98	8.36	9.43	13.56	VE	34[2]
						AE	34[2]

*Due to map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted.
**North American Vertical Datum of 1988
[1]Including stillwater elevations and effects of wave setup
[2]Wave runup elevation

Users of the FIRM should also be aware that coastal flood elevations are provided in Table 5 "Summary of Coastal Stillwater Elevations" in this report. If the elevation on the FIRM is higher than the elevation shown in this table, a wave height, wave runup, and/or wave setup component likely exists, in which case, the higher elevation should be used for construction and/or floodplain management purposes.

As defined in the July 1989 *Guidelines and Specifications for Wave Elevation Determination and V Zone Mapping*, the coastal high hazard area (Zone VE) is the area where wave action and/or high velocity water can cause structural damage (*Guidelines and Specifications for Wave Elevation Determination and V-Zone Mapping*, FEMA, 1989). It is designated on the FIRM as the most landward of the following three points:

1) The point where the 3.0 ft or greater wave height could occur;

2) The point where the eroded ground profile is 3.0 ft or more below the maximum runup elevation; or

3) The primary frontal dune as defined in the NFIP regulations.

These three points are used to locate the inland limit of the coastal high hazard area to ensure that adequate insurance rates apply and appropriate construction standards are used, should local agencies permit building in this area.

The Limit of Moderate Wave Action (LiMWA) was delineated in accordance with FEMA Procedure Memorandum 50 (2008). In coastal areas, Zone AE may be subdivided by a limit of moderate wave action boundary at the landward extent of the propagation of waves higher than 1.5 feet. Damages to structures from wave heights between 1.5 and 3 feet are similar to, but less severe than, those in areas where wave heights are greater than 3 feet, typically designated as Zone VE on the FIRM. Damages to structures from wave heights less than 1.5 feet are more similar to those in riverine or lacustrine floodplains. The inland limit of the area affected by waves greater than 1.5 feet is called the Limit of Moderate Wave Action (LiMWA).

3.4 Vertical Datum

Bench marks cataloged by the NGS and entered into the NSRS vary widely in vertical stability classification. NSRS vertical stability classifications are as follows:

- Stability A: Monuments of the most reliable nature, expected to hold position/elevation well (e.g., mounted in bedrock)

- Stability B: Monuments which generally hold their position/elevation well (e.g., concrete bridge abutment)

- Stability C: Monuments which may be affected by surface ground movements (e.g., concrete monument below frost line)

- Stability D: Mark of questionable or unknown vertical stability (e.g., concrete monument above frost line, or steel witness post)

In addition to NSRS bench marks, the FIRM may also show vertical control monuments established by a local jurisdiction; these monuments will be shown on the FIRM with the appropriate designations. Local monuments will only be placed on the FIRM if the community has requested that they be included, and if the monuments meet the aforementioned NSRS inclusion criteria.

All FISs and FIRMs are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Previously, the standard vertical datum in use for newly created or revised FISs and FIRMs was the National Geodetic Vertical Datum (NGVD 29). With the finalization of the North American Vertical Datum of 1988 (NAVD 88), FIS reports and FIRMs are typically being prepared using NAVD 88 as the referenced vertical datum.

The conversion factor between NGVD 29 and NAVD 88 for Rockingham County is -.7 ft. Elevation 0 NGVD 29 is elevation -0.7 NAVD 88.

Flood elevations shown in this FIS report and on the FIRM for the following 14 coastal communities are referenced to NAVD 88: Greenland, Hampton, New Castle, Newfields, Newington, Newmarket, North Hampton, Portsmouth, Rye, Seabrook, Seabrook Beach Village District, and the Village District of Little Boar's Head. Structure and ground elevations in these communities must, therefore, be referenced to NAVD88.

Flood elevations shown in this FIS report and on the FIRMs for Exeter, Hampton Falls, and Stratham are referenced to NAVD88 on updated panels and streams, and remain in NGVD29 for panels that are not updated for the January 29, 2021 effective date (see the Index for effective dates per panel).

Flood elevations shown in this FIS report and on the FIRM for the 24 remaining, interior communities in Rockingham County, including Atkinson, Auburn, Brentwood, Candia, Chester, Danville, Deerfield, Derry, East Kingston, Epping, Fremont, Hampstead, Kensington, Kingston, Londonderry, Newton, Northwood, Nottingham, Plaistow, Raymond, Sandown, Salem, South Hampton, and Windham are referenced to NGVD29. Structure and ground elevations in these communities must, therefore, be referenced to NGVD 29. It is important to note that adjacent communities may be referenced to NAVD 88. This may result in differences in base flood elevations across the corporate limits between the communities.

A summary of the vertical datum reference by town in Rockingham County is provided in Table 9, "Vertical Datum Reference by Community."

TABLE 9 – VERTICAL DATUM REFERENCE BY COMMUNITY

Community Name	Vertical Datum Reference
Atkinson	NGVD 29
Auburn	NGVD 29
Brentwood	NGVD 29
Candia	NGVD 29
Chester	NGVD 29
Danville	NGVD 29
Deerfield	NGVD 29
Derry	NGVD 29
East Kingston	NGVD 29
Epping	NGVD 29
Exeter	NGVD 29 & NAVD 88
Fremont	NGVD 29
Greenland	NAVD 88
Hampstead	NGVD 29
Hampton	NAVD 88
Hampton Falls	NGVD 29 & NAVD 88
Kensington	NGVD 29
Kingston	NGVD 29
Little Boar's Head	NAVD 88
Londonderry	NGVD 29
New Castle	NAVD 88
Newfields	NAVD 88
Newington	NAVD 88
Newmarket	NAVD 88
Newton	NGVD 29
North Hampton	NAVD 88
Northwood	NGVD 29
Nottingham	NGVD 29
Plaistow	NGVD 29
Portsmouth	NAVD 88
Raymond	NGVD 29
Rye	NAVD 88
Sandown	NGVD 29
Salem	NGVD 29
Seabrook	NAVD 88
Seabrook Beach Village District	NAVD 88
South Hampton	NGVD 29
Stratham	NGVD 29 & NAVD 88
Windham	NGVD 29

For more information on NAVD 88, see Converting the National Flood Insurance Program to the North American Vertical Datum of 1988, FEMA Publication FIA-20/June 1992, or contact the Vertical Network Branch, National Geodetic Survey, Coast and Geodetic Survey, National Oceanic and Atmospheric Administration, Rockville, Maryland 20910 (Internet address http://www.ngs.noaa.gov).

4.0 FLOODPLAIN MANAGEMENT APPLICATIONS

The NFIP encourages State and local governments to adopt sound floodplain management programs. To assist in this endeavor, each FIS provides 1 percent annual chance floodplain data, which may include a combination of the following: 10-, 2-, 1-, and 0.2-percent annual chance flood elevations; delineations of the 1 percent annual chance and 0.2 percent annual chance floodplains; and 1 percent annual chance floodway. This information is presented on the FIRM and in many components of the FIS, including Flood Profiles, Floodway Data tables, and Summary of Stillwater Elevation tables. Users should reference the data presented in the FIS as well as additional information that may be available at the local community map repository before making flood elevation and/or floodplain boundary determinations.

4.1 Floodplain Boundaries

To provide a national standard without regional discrimination, the 1-percent annual chance (100-year) flood has been adopted by FEMA as the base flood for floodplain management purposes. The 0.2-percent annual chance (500-year) flood is employed to indicate additional areas of flood risk in the county. For the streams studied in detail, the 1 percent annual chance and 0.2 percent annual chance floodplain boundaries have been delineated using the flood elevations determined at each cross section.

Pre-countywide Analysis

Between the cross sections, the boundaries were interpolated using topographic maps (State of New Hampshire, 1970; USGS, 1956, 1966, 1973, 1974, 1977, 1981, 1985; James W. Sewall Company, 1976, 1977, 1978, 1979; Southeastern New Hampshire Regional Planning Commission, New Hampshire, August 1974; Avis Airmap, 1977; Southeastern New Hampshire Regional Planning Commission, Concord, New Hampshire, July 1975; and Underwood Engineers) and soil survey maps (U.S. Department of Agriculture, 1980, 1981, 1983, and 1986).

For the streams studied by approximate methods, the 1 percent annual chance floodplain boundaries were delineated using a combination of the following: previously printed Flood Hazard Boundary Maps (U.S. Department of Housing and Urban Development, 1974, 1975, 1976, 1977; FEMA, 1986); previously printed FISs (FEMA, 1981 and 1988); topographic maps (USGS, 1953, 1956, 1966, 1968, 1973, 1974, and 1981; James W. Sewall Company, 1976, 1977, 1979; S.N.H.R.P.C., 1975, 1976); SCS Flood Prone Area Map (U.S. Department of Agriculture, 1974); and soil survey map (U.S. Department of Agriculture, 1983).

The 1 percent annual chance and 0.2 percent annual chance floodplain boundaries are shown on the FIRM (Exhibit 2). On this map, the 1 percent annual chance floodplain boundary corresponds to the boundary of the areas of special flood hazards (Zones A and AE), and the 0.2 percent annual chance floodplain boundary corresponds to the boundary of areas of moderate flood hazards. In cases where the 1 percent annual chance and 0.2 percent annual chance floodplain boundaries are close together, only the 1 percent annual chance floodplain boundary has been shown. Small areas within the floodplain boundaries may lie above the flood elevations but cannot be shown due to limitations of the map scale and/or lack of detailed topographic data.

For the streams studied by approximate methods, only the 1 percent annual chance floodplain boundary is shown on the FIRM (Exhibit 2).

1 percent annual chance flood data elevations are shown in Table 10, "1% Annual Chance Flood Data."

FLOODING SOURCE		RIVER CHANNEL				1% ANNUAL CHANCE WATER-SURFACE ELEVATIONS (FEET NGVD)
CROSS SECTION	DISTANCE[1] (FEET)	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	STREAM-BED ELEVATION (FT. NGVD)	
Hog Hill Brook						
A	20	125	603	1.1	127.2	137.4
B	1,540	140	682	1.0	128.0	137.9
C	1,600	180	713	1.0	129.4	138.0
D	2,580	50	93	7.3	140.7	143.6
E	2,650	126	761	0.9	142.5	154.3
F	2,800	147	531	1.3	145.6	154.3
G	2,850	200	220	3.1	149.1	154.3
H	4,000	73	125	3.3	149.8	154.5
I	4,390	30	54	7.6	161.1	164.4
J	4,460	214	436	0.9	164.1	168.6
K	5,400	57	84	4.9	168.6	172.0
L	6,100	67	148	2.8	174.7	178.5
M	7,820	147	355	1.2	176.2	181.5
N	8,910	289	553	0.7	178.3	181.8
O	8,980	95	421	0.9	180.3	188.5

'Distance in feet above Town of Atkinson corporate limits

TABLE 10

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH

(ALL JURISDICTIONS)

1% ANNUAL CHANCE FLOOD DATA

HOG HILL BROOK

2005 Countywide Analyses

No remapping was conducted in 2005.

The January 29, 2021 Countywide Revision

For streams studied in detail, 1-percent and 0.2-percent annual chance floodplain boundaries were delineated using the flood elevations determined at each cross section. Between cross sections, the boundaries were interpolated based on 2-foot contour interval topography from the 2011 LiDAR mission discussed in Section 2.1. The LiDAR was also utilized to support the basic Zone A modeling and delineations, as well as the redelineation of hydraulic analyses from previous studies.

For tidal areas without wave action, the 1 percent annual chance and 0.2 percent annual chance flood boundaries were also delineated using the 2011 LiDAR. For the tidal areas with wave action, the flood boundaries were delineated using the elevations determined at each transect; between transects, the boundaries were interpolated using engineering judgment, land-cover data, and the topographic maps referenced above. The 1 percent annual chance floodplain was divided into whole-foot elevation zones based on average wave envelope elevation in that zone. Where the map scale did not permit these zones to be delineated at one-foot intervals, larger increments were used.

4.2 Floodways

Encroachment on floodplains, such as structures and fill, reduces flood-carrying capacity, increases flood heights and velocities, and increases flood hazards in areas beyond the encroachment itself. One aspect of floodplain management involves balancing the economic gain from floodplain development against the resulting increase in flood hazard. For purposes of the NFIP, a floodway is used as a tool to assist local communities in this aspect of floodplain management. Under this concept, the area of the 1 percent annual chance floodplain is divided into a floodway and a floodway fringe. The floodway is the channel of a stream, plus any adjacent floodplain areas, that must be kept free of encroachment so that the 1 percent annual chance flood can be carried without substantial increases in flood heights. Minimum federal standards limit such increases to 1.0 foot, provided that hazardous velocities are not produced. The floodways in this FIS are presented to local agencies as minimum standards that can be adopted directly or that can be used as a basis for additional floodway studies.

The floodways presented in this FIS were computed for certain stream segments on the basis of equal conveyance reduction from each side of the floodplain. Floodway widths were computed at cross sections. Between cross sections, the floodway boundaries were interpolated. The results of the floodway computations are tabulated for selected cross sections (Table 11). The computed floodways are shown on the FIRM (Exhibit 2). In cases where the floodway and 1 percent annual chance floodplain boundaries are either close together or collinear, only the floodway boundary is shown.

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NGVD29)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Beaver Brook								
A	13.926	135/25[2]	707	4.3	152.0	152.0	152.5	0.5
B	13.947	50/30[2]	415	7.4	154.7	154.7	154.7	0.0
C	14.037	85/65[2]	553	5.6	156.5	156.5	157.5	1.0
D	14.738	85/55[2]	573	5.4	163.5	163.5	164.1	0.6
E	14.942	180/120[2]	1,423	2.2	166.9	166.9	167.0	0.1
F	15.646	210/20[2]	1,266	2.4	167.8	167.8	168.8	1.0
G	15.990	50/20[2]	463	6.3	172.6	172.6	172.6	0.0
H	16.417	165/25[2]	1,105	2.6	175.4	175.4	175.9	0.5
I	17.057	160	663	4.2	176.7	176.7	177.7	1.0
J	17.964	50	327	8.2	192.1	192.1	193.1	1.0
K	18.993	110	821	3.3	209.1	209.1	209.1	0.0
L	20.017	50	444	6.1	210.0	210.0	211.0	1.0
M	20.482	90	634	4.2	213.5	213.5	214.2	0.7
N	21.305	80	617	3.3	219.2	219.2	220.2	1.0
0	21.799	195	560	3.7	219.9	219.9	220.6	0.7
P	22.802	260	1,565	1.3	226.0	226.0	227.0	1.0
Q	23.392	40	341	6.0	230.9	230.9	230.9	0.0
R	23.816	300	1,344	1.5	231.8	231.8	232.7	0.9
S	24.233	110	606	3.4	235.9	235.9	236.5	0.6
T	24.694	180	910	2.3	238.0	238.0	238.9	0.9
U	25.075	100	654	2.2	241.2	241.2	241.3	0.1
V	25.546	100	598	2.4	242.7	242.7	243.4	0.7
W	25.789	127	962	1.5	244.4	244.4	245.1	0.7
X	26.233	230	2,276	0.6	248.0	248.0	248.9	0.9
Y	26.648	300	2,677	0.2	248.0	248.0	248.9	0.9
Z	26.870	350	1,801	0.2	248.0	248.0	248.9	0.9

[1]Miles above confluence with Merrimack River
[2]Width/width within county boundary

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH

(ALL JURISDICTIONS)

FLOODWAY DATA

BEAVER BROOK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NGVD29)			
CROSS SECTION	DISTANCE	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Beaver Brook (continued)								
AA	27.244[1]	80	437	1.0	248.1	248.1	248.9	0.8
AB	27.580[1]	24	55	7.8	253.6	253.6	253.8	0.2
AC	27.652[1]	32	112	3.8	263.7	263.7	263.9	0.2
AD	27.838[1]	30	59	7.3	282.0	282.0	282.1	0.1
Black Brook								
A	0.400[2]	115	288	0.9	214.0	212.0[4]	212.8	0.8
B	1.000[2]	30	90	2.9	216.4	216.4	216.8	0.4
C	1.545[2]	20	43	6.2	257.2	257.2	257.2	0.0
D	1.737[2]	20	19	4.7	264.5	264.5	264.5	0.0
E	2.095[2]	30	17	5.3	281.5	281.5	281.5	0.0
F	2.369[2]	20	14	6.4	298.6	298.6	298.6	0.0
G	3.176[2]	25	23	3.9	321.0	321.0	321.0	0.0
Bryant Brook								
A	660[3]	27	59	6.0	47.8	47.8	48.8	1.0
B	1,370[3]	27	41	8.7	67.3	67.3	67.3	0.0
C	1,760[3]	15	37	9.6	73.3	73.3	73.7	0.4
D	2,815[3]	228	473	0.8	74.7	74.7	75.7	1.0
E	4,010[3]	96	193	1.8	76.3	76.3	77.3	1.0
F	5,955[3]	80	240	1.5	78.7	78.7	79.7	1.0
G	6,810[3]	238	395	0.9	79.3	79.3	80.3	1.0

[1]Miles above confluence with Merrimack River
[2]Miles above confluence with Beaver Brook
[3]Feet above confluence with Little River No. 3
[4]Elevation computed without consideration of backwater effects from Beaver Brook

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH

(ALL JURISDICTIONS)

FLOODWAY DATA

BEAVER BROOK - BLACK BROOK - BRYANT BROOK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NGVD29)			
CROSS SECTION	DISTANCE	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Cohas Brook								
A	0.000[1]	30	155	6.3	227.3	227.3	228.3	1.0
B	0.312[1]	30	120	8.2	233.7	233.7	234.1	0.4
C	0.700[1]	50	202	4.9	245.0	245.0	246.0	1.0
D	1.032[1]	40	163	6.0	249.4	249.4	250.1	0.7
E	1.350[1]	80	348	2.8	259.7	259.7	260.4	0.7
Cunningham Brook								
A	0.155[2]	31	149	2.5	218.9	218.9	218.9	0.0
B	0.514[2]	24	55	6.7	251.6	251.6	252.1	0.5
C	1.040[2]	276	833	0.4	296.0	296.0	297.0	1.0
Drew Brook								
A	0.100[3]	170	974	0.4	206.8	206.8	207.8	1.0
B	0.425[3]	140	854	0.4	207.6	207.6	208.0	0.4
C	0.705[3]	65	376	0.9	208.9	208.9	208.9	0.0
D	1.043[3]	40	165	2.1	209.2	209.2	209.4	0.2
E	1.800[3]	70	129	2.7	213.8	213.8	214.0	0.2

[1]Miles above county boundary
[2]Miles above confluence with Drew Brook
[3]Miles above confluence with Island Pond

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH

(ALL JURISDICTIONS)

FLOODWAY DATA

COHAS BROOK - CUNNINGHAM BROOK - DREW BROOK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NGVD29)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Dudley Brook								
A	2,198	56	228	2.6	82.6	82.6	83.5	0.9
B	2,375	101	967	0.6	89.7	89.7	89.7	0.0
C	7,475	57	250	2.0	89.8	89.8	90.0	0.2
D	7,644	56	236	2.1	89.8	89.8	90.0	0.2
E	7,720	24	57	8.8	92.7	92.7	92.7	0.0
F	7,847	53	294	1.7	94.1	94.1	94.2	0.1
G	9,237	74	335	1.5	94.2	94.2	94.8	0.6
H	12,277	255	591	0.9	96.0	96.0	96.7	0.7
I	18,627	164	322	1.0	102.0	102.0	102.9	0.9
J	20,007	24	78	3.9	106.7	106.7	106.8	0.1
K	20,237	32	128	2.4	107.1	107.1	108.1	1.0
L	20,439	15	87	3.5	107.5	107.5	108.5	1.0
M	20,487	12	77	4.0	107.6	107.6	108.6	1.0

[1]Feet above Town of Brentwood corporate limits

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH

(ALL JURISDICTIONS)

FLOODWAY DATA

DUDLEY BROOK

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NGVD29)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Exeter River								
A	0	213	416	11.6	11.8	11.8	11.8	0.0
B	175	120	647	7.5	13.8	13.8	13.8	0.0
C	325	93	965	5.0	22.0	22.0	22.1	0.1
D	395	135	1,920	2.5	30.4	30.4	30.5	0.1
E	598	70	938	5.1	31.1	31.1	31.1	0.0
F	2338	119	1,634	3.0	31.7	31.7	32.2	0.5
G	2451	99	1,656	2.9	31.7	31.7	32.2	0.5
H	3681	549	4,257	1.0	31.7	31.7	32.5	0.8
I	6,421	820	5,696	0.7	32.0	32.0	32.9	0.9
J	9,381	639	5,632	0.7	32.4	32.4	33.4	1.0
K	15,881	956	7,956	0.5	32.7	32.7	33.7	1.0
L	19,231	1,218	6,205	0.6	32.9	32.9	33.9	1.0
M	23,829	142	1,500	2.5	33.5	33.5	34.5	1.0
N	23,940	73	860	4.3	34.6	34.6	34.8	0.2
O	25,140	196	1,992	1.9	35.3	35.3	36.1	0.8
P	26,280	351	2,433	1.5	35.6	35.6	36.6	1.0
Q	30,590	546	5,019	0.7	36.0	36.0	37.0	1.0
R	30,709	391	2,811	1.3	36.2	36.2	37.1	0.9
S	31,929	913	6,629	0.6	36.4	36.4	37.3	0.9
T	34,759	109	396	9.5	37.5	37.5	37.5	0.0
U	35,379	92	1,058	3.5	41.1	41.1	41.8	0.7
V	35,504	70	778	4.8	42.9	42.9	43.2	0.3
W	37,789	73	776	4.8	45.5	45.5	46.5	1.0
X	39,510	100	436	8.6	50.6	50.6	51.4	0.8

[1]Feet above confluence with Squamscott River.

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH
(ALL JURISDICTIONS)

FLOODWAY DATA

EXETER RIVER

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NGVD29)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Exeter River								
Y	39,608	81	867	4.3	59.4	59.4	59.4	0.0
Z	39,776	257	2,210	1.7	65.9	65.9	66.9	1.0
AA	41,626	135	1,276	2.9	66.1	66.1	67.0	0.9
AB	42,276	390	2,386	1.4	66.3	66.3	67.2	0.9
AC	52,603	274	1,215	2.7	67.2	67.2	67.9	0.7
AD	56,283	350	3,357	0.9	68.7	68.7	69.6	0.9
AE	58,143	99	508	5.9	70.0	70.0	70.5	0.5
AF	58,315	59	327	9.2	70.3	70.3	70.7	0.4
AG	61,175	97	1,104	2.7	73.7	73.7	74.0	0.3
AH	65,655	88	682	4.4	75.4	75.4	75.8	0.4
AI	66,895	67	555	5.4	76.7	76.7	77.0	0.3
AJ	69,895	74	621	4.8	80.3	80.3	80.6	0.3
AK	71,490	73	424	7.1	83.0	83.0	83.4	0.4
AL	72,560	43	233	12.9	91.4	91.4	92.0	0.6
AM	72,763	70	274	11.0	100.6	100.6	100.6	0.0
AN	72,842	70	467	6.4	104.5	104.5	104.6	0.1
AO	72,887	74	503	6.0	104.7	104.7	104.8	0.1
AP	73,031	36	297	10.1	104.7	104.7	104.8	0.1
AQ	73,165	164	1,218	2.5	107.2	107.2	107.2	0.0
AR	77,960	190	1,009	3.0	116.0	116.0	117.0	1.0
AS	78,530	64	393	7.7	120.4	120.4	120.4	0.0
AT	78,701	52	760	4.0	129.7	129.7	129.7	0.0

[1]Feet above confluence with Squamscott River

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH
(ALL JURISDICTIONS)

FLOODWAY DATA

EXETER RIVER

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NGVD29)			
CROSS SECTION	DISTANCE'	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Exeter River								
AU	78,751	89	1,468	2.1	133.7	133.7	133.7	0.0
AV	78,936	136	1,489	2.0	133.7	133.7	133.8	0.1
AW	80,076	109	743	3.9	133.9	133.9	134.0	0.1
AX	80,323	109	760	3.8	134.0	134.0	134.1	0.1
AY	80,373	219	1,519	1.9	134.2	134.2	134.3	0.1
AZ	80,360	219	1,546	1.9	135.3	135.3	135.3	0.0
BA	82,740	275	2,762	1.0	135.5	135.5	135.5	0.0
BB	84,960	185	1,684	1.9	135.6	135.6	135.8	0.2

'Feet above confluence with Squamscott River

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH

(ALL JURISDICTIONS)

FLOODWAY DATA

EXETER RIVER

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NGVD29)			
CROSS SECTION	DISTANCE	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Flatrock Brook								
A	0.209[1]	35	140	5.0	165.3	165.3	165.3	0.0
B	0.447[1]	68	272	2.6	169.1	169.1	170.0	0.9
C	0.737[1]	17	130	5.4	182.4	182.4	182.4	0.0
D	0.969[1]	37	180	2.9	182.9	182.9	183.9	1.0
E	1.325[1]	21	61	8.6	232.7	232.7	232.8	0.1
F	1.800[1]	24	89	4.0	240.1	240.1	240.8	0.7
Golden Brook								
A	3.705[2]	75	349	2.0	139.8	139.8	139.9	0.1
B	4.880[2]	100	524	1.4	151.4	151.4	152.3	0.9
C	5.728[2]	110	641	1.2	156.2	156.2	156.3	0.1
D	7.390[2]	21	57	6.7	177.9	177.9	177.9	0.0
E	7.962[2]	25	51	7.5	188.8	188.8	189.1	0.3
F	8.535[2]	21	65	5.9	208.4	208.4	208.7	0.3
G	8.649[2]	11	102	3.7	221.4	221.4	221.6	0.2
Hidden Valley Brook								
A	0.200[3]	17	81	3.6	210.2	208.4[4]	209.1	0.7
B	0.500[3]	13	93	3.1	218.0	218.0	218.0	0.0
C	0.900[3]	15	38	7.5	240.1	240.1	240.3	0.2
D	1.125[3]	20	51	4.1	249.1	249.1	249.5	0.4
E	1.383[3]	75	168	1.0	251.2	251.2	252.1	0.9
F	1.591[3]	40	63	2.7	267.7	267.7	267.9	0.2
G	2.073[3]	17	48	4.4	276.0	276.0	277.0	1.0

[1]Miles above confluence with Shadow Lake
[2]Miles above mouth
[3]Miles above confluence with Beaver Brook
[4]Elevation computed without consideration of backwater effects from Beaver Brook

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH

(ALL JURISDICTIONS)

FLOODWAY DATA

FLATROCK BROOK - GOLDEN BROOK - HIDDEN VALLEY BROOK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NGVD29)			
CROSS SECTION	DISTANCE	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Homes Brook								
A	0.083[1]	18	91	4.0	241.0	239.4[3]	240.1	0.7
B	0.347[1]	16	81	4.5	243.2	243.2	244.0	0.8
C	0.620[1]	18	84	4.4	250.6	250.6	251.3	0.7
D	0.758[1]	20	92	4.0	252.8	252.8	253.7	0.9
Kelly Brook								
A	575[2]	25	114	4.4	96.4	96.4	97.4	1.0
B	1,160[2]	40	122	4.1	98.2	98.2	98.9	0.7
C	4,000[2]	65	697	0.7	111.9	111.9	112.0	0.1
D	5,410[2]	40	328	1.5	111.9	111.9	112.1	0.2
E	6,930[2]	20	160	3.1	116.3	116.3	117.1	0.8
F	7,490[2]	30	143	3.5	116.7	116.7	117.6	0.9
G	8,880[2]	45	104	4.8	123.5	123.5	124.1	0.6
H	9,135[2]	30	76	6.5	125.6	125.6	125.9	0.3

'Miles above confluence with Beaver Brook
[2]Feet above confluence with Little River No. 3
[3]Elevation computed without consideration of backwater effects from Beaver Brook

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH

(ALL JURISDICTIONS)

FLOODWAY DATA

HORNES BROOK - KELLY BROOK

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Lamprey River (Town of Newmarket)								
A	0	86	597	14.9	10.3	10.3	10.3	0.0
B	36	140	3068	2.9	33.5	33.5	34.5	1.0
C	206	139	3494	2.6	33.6	33.6	34.6	1.0
D	247	92	1552	5.8	33.6	33.6	34.5	0.9
E	310	68	1406	6.4	34.6	34.6	35.4	0.8
F	345	132	2082	4.3	34.9	34.9	35.9	1.0
G	546	135	3039	2.9	35.1	35.1	36.1	1.0
H	754	195	4697	1.9	35.2	35.2	36.1	0.9
I	1764	203	4276	2.1	35.3	35.3	36.2	0.9
J	1947	277	5516	1.6	35.3	35.3	36.2	0.9
K	2885	385	7368	1.2	35.4	35.4	36.3	0.9

[1]Feet above MacCallen Dam.

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH (ALL JURISDICTIONS)

FLOODWAY DATA

LAMPREY RIVER (TOWN OF NEWMARKET)

LOCATION		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NGVD29)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Lamprey River								
A	0	119	1,319	4.3	95.4	95.4	96.4	1.0
B	5,550	356	2,746	1.8	97.9	97.9	98.9	1.0
C	10,960	97	1,267	3.9	100.1	100.1	101.1	1.0
D	16,510	261	2,436	2.0	102.3	102.3	103.3	1.0
E	19,310	199	2,339	2.1	102.8	102.8	103.8	1.0
F	19,440	414	3,926	1.3	103.0	103.0	104.0	1.0
G	29,570	498	3,886	1.3	105.6	105.6	106.6	1.0
H	32,620	112	1,233	4.0	107.2	107.2	108.2	1.0
I	36,130	100	1,064	4.6	109.5	109.5	110.5	1.0
J	36,900	138	1,462	3.4	110.4	110.4	111.4	1.0
K	37,240	149	1,451	3.4	110.8	110.8	111.8	1.0
L	37,980	149	2,251	2.2	111.5	111.5	111.6	0.1
M	38,220	102	1,157	4.3	112.3	112.3	113.3	1.0
N	41,620	390	3,465	1.4	113.5	113.5	114.5	1.0
O	44,620	105	1,119	4.2	115.6	115.6	116.6	1.0
P	54,730	112	1,400	3.4	138.0	138.0	139.0	1.0
Q	57,290	163	1,930	2.5	138.8	138.8	139.8	1.0
R	57,660	199	2,052	2.0	138.9	138.9	139.9	1.0
S	57,740	198	1,034	4.0	138.9	138.9	139.9	1.0
T	58,440	161	1,859	2.3	147.6	147.6	148.6	1.0
U	64,620	123	1,045	4.0	153.0	153.0	154.0	1.0
V	66,900	128	1,256	3.3	155.4	155.4	156.4	1.0
W	69,780	86	817	6.5	163.7	163.7	164.7	1.0
X	71,330	137	1,322	4.0	165.7	165.7	166.7	1.0
Y	71,470	99	981	5.4	166.3	166.3	167.3	1.0

[1]Feet above county boundary.

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH
(ALL JURISDICTIONS)

FLOODWAY DATA

LAMPREY RIVER

LOCATION		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NGVD29)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Lamprey River (continued)								
Z	77,180	227	2,147	2.5	167.8	167.8	168.8	1.0
AA	77,760	113	502	10.5	177.1	177.1	178.1	1.0
AB	77,810	120	501	10.6	178.6	178.6	179.6	1.0
AC	78,190	156	1,197	4.4	181.0	181.0	182.0	1.0
AD	83,080	159	1,658	3.19	184.7	184.7	185.7	1.0
AE	83,910	102	1,277	4.14	185.9	185.9	186.9	1.0
AF	84,610	107	1,149	4.61	186.4	186.4	187.4	1.0
AG	84,830	279	4,359	1.21	190.1	190.1	191.1	1.0
AH	89,830	205	2,666	1.98	190.3	190.3	191.3	1.0
AI	95,610	270	3,362	1.30	190.8	190.8	191.8	1.0
AJ	97,110	51	635	6.88	193.1	193.1	194.1	1.0
AK	97,380	144	1,411	3.10	195.8	195.8	196.8	1.0
AL	98,230	177	1,490	2.93	196.4	196.4	197.4	1.0
AM	101,400	317	1,560	2.80	200.6	200.6	201.6	1.0
AN	102,430	81	684	6.39	202.6	202.6	203.6	1.0
AO	105,160	81	787	5.55	206.7	206.7	207.7	1.0
AP	107,920	138	1,629	2.68	207.9	207.9	208.9	1.0
AQ	110,110	237	2,271	1.45	211.7	211.7	212.7	1.0
AR	110,410	134	1,568	2.10	213.0	213.0	214.0	1.0
AS	113,530	96	1,041	3.17	214.4	214.4	215.4	1.0
AT	115,130	150	994	3.32	216.4	216.4	217.4	1.0
AU	116,790	203	2,305	1.43	216.7	216.7	217.7	1.0
AV	119,400	1,407	9,085	0.36	216.8	216.8	217.8	1.0

[1]Feet above county boundary.

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH (ALL JURISDICTIONS)

FLOODWAY DATA

LAMPREY RIVER

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NGVD29)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Little Cohas Brook								
A	0.141	20	52	9.2	200.4	200.4	200.4	0.0
B	0.547	30	112	4.3	212.1	212.1	212.2	0.1
C	0.678	30	73	6.6	229.2	229.2	229.2	0.0
D	0.900	40	56	6.9	242.7	242.7	242.7	0.0
E	1.165	180	720	0.5	261.1	261.1	261.1	0.0
F	1.228	630	3,062	0.1	263.7	263.7	263.7	0.0
G	1.775	105	487	0.8	263.7	263.7	263.7	0.0
H	2.365	30	175	1.8	264.3	264.3	264.4	0.1
I	2.717	300	396	0.8	264.3	264.3	265.1	0.8
J	3.405	20	25	6.8	306.8	306.8	306.8	0.0

'Miles above Industrial Drive

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH

(ALL JURISDICTIONS)

FLOODWAY DATA

LITTLE COHAS BROOK

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NGVD29)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Little River No. 1								
A	400	195	1,679	0.4	31.7	28.8[2]	28.8	0.0
B	610	80	803	0.8	31.7	28.8[2]	28.8	0.0
C	2,460	70	615	1.0	31.7	28.8[2]	28.9	0.1
D	2,604	99	839	0.7	31.7	28.9[2]	29.0	0.1
E	4,104	29	183	3.4	31.7	29.0[2]	29.1	0.1
F	5,104	44	351	1.8	31.7	29.0[2]	29.8	0.8
G	5,234	214	1,118	0.6	31.7	29.4[2]	30.2	0.8
H	7,634	76	504	1.2	31.7	29.7[2]	30.5	0.8
I	7,934	76	696	0.9	31.7	29.8[2]	30.7	0.9
J	8,069	78	287	2.2	31.7	30.6[2]	31.2	0.6
K	9,219	122	427	1.5	31.7	31.5[2]	32.2	0.7
L	10,169	164	800	0.8	31.7	31.7	32.4	0.7
M	10,246	21	128	4.9	31.7	31.7	32.4	0.7
N	10,566	80	430	1.5	32.4	32.4	33.0	0.6
O	11,866	32	173	3.6	32.7	32.7	33.4	0.7
P	12,666	55	87	7.2	40.4	40.4	40.7	0.3
Q	12,799	205	1,221	0.5	47.5	47.5	47.6	0.1

[1]Feet above confluence with Exeter River
[2]Elevation computed without consideration of backwater effects from Exeter River

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH

(ALL JURISDICTIONS)

FLOODWAY DATA

LITTLE RIVER NO. 1

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Little River No. 2								
A	3,048	67	304	0.7	9.3	9.3	9.4	0.1
B	5,048	*	78	2.9	9.6	9.6	10.1	0.5
C	5,185	*	59	3.8	10.0	10.0	10.4	0.4
D	5,385	*	32	7.2	11.8	11.8	11.8	0.0
E	5,490	*	31	7.3	13.8	13.8	14.0	0.2
F	5,780	*	25	9.0	20.9	20.9	21.0	0.1
G	6,420	*	31	7.4	26.3	26.3	26.3	0.0
H	6,495	*	32	7.2	30.9	30.9	31.0	0.1
I	6,561	75	410	0.6	34.6	34.6	34.8	0.2
J	6,771	*	25	9.0	34.8	34.8	34.8	0.0
K	6,867	*	49	4.6	38.3	38.3	38.3	0.0

'Feet above downstream dam in Town of North Hampton
*Floodway coincident with channel banks

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH

(ALL JURISDICTIONS)

FLOODWAY DATA

LITTLE RIVER NO. 2

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NGVD29)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Little River No. 3								
A	290	40	213	6.0	39.7	39.7	40.4	0.7
B	1,600	30	281	4.5	42.2	42.2	42.9	0.7
C	3,110	119	614	1.8	43.1	43.1	44.1	1.0
D	3,265	85	574	1.9	43.7	43.7	44.5	0.8
E	4,640	91	285	3.8	45.0	45.0	45.9	0.9
F	5,035	42	243	4.4	47.4	47.4	47.5	0.1
G	5,340	35	205	5.2	49.9	49.9	49.9	0.0
H	7,490	32	197	5.5	54.6	54.6	55.1	0.5
I	8,704	40	120	9.0	58.4	58.4	58.4	0.0
J	10,030	135	850	0.9	60.1	60.1	61.1	1.0
K	10,480	60	327	2.4	61.8	61.8	62.6	0.8
L	11,450	145	880	0.9	61.9	61.9	62.8	0.9
M	12,660	70	278	2.9	62.6	62.6	63.4	0.8
N	14,850	48	250	3.2	64.7	64.7	65.4	0.7
O	15,730	53	163	4.9	68.3	68.3	69.1	0.8
P	16,850	20	161	4.9	81.8	81.8	81.8	0.0
Q	17,770	39	91	8.7	86.4	86.4	86.4	0.0
R	19,420	33	142	5.6	93.3	93.3	93.8	0.5
S	20,690	70	314	2.5	95.2	95.2	96.0	0.8
T	21,970	34	153	5.2	96.3	96.3	97.1	0.8
U	23,066	50	254	1.9	102.9	102.9	102.9	0.0
V	25,410	51	326	1.5	103.1	103.1	103.5	0.4
W	27,555	58	225	1.5	103.5	103.5	104.2	0.7
X	28,240	22	127	2.6	106.9	106.9	106.9	0.0

'Feet above New Hampshire-Massachusetts State boundary

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH

(ALL JURISDICTIONS)

FLOODWAY DATA

LITTLE RIVER NO. 3

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NGVD29)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Nesenkeag Brook								
A	0.278	150	228	3.3	178.7	178.7	179.4	0.7
B	0.730	20	37	5.7	190.9	190.9	191.1	0.2
C	1.262	20	62	3.4	196.1	196.1	196.6	0.5
D	1.665	30	33	6.4	225.2	225.2	225.2	0.0
E	1.900	30	89	2.4	229.6	229.6	229.8	0.2
F	2.245	30	30	7.0	251.9	251.9	251.9	0.0
G	3.247	30	210	1.0	271.7	271.7	272.6	0.9
H	3.381	20	123	1.7	273.6	273.6	273.6	0.0
I	3.533	10	137	1.5	289.6	289.6	289.6	0.0

‘Miles above county boundary

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH

(ALL JURISDICTIONS)

FLOODWAY DATA

NESENKEAG BROOK

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Piscassic River								
A	4,630	68	341	1.1	91.4	91.4	92.4	1.0
B	6,530	30	177	2.1	94.2	94.2	95.2	1.0
C	7,120	26	121	3.1	97.9	97.9	98.9	1.0
D	9,575	95	305	1.2	100.1	100.1	101.1	1.0

[1]Feet above Ice Pond Dam

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH

(ALL JURISDICTIONS)

FLOODWAY DATA

PISCASSIC RIVER

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NGVD29)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Policy Brook								
A	0	50	160	4.1	124.0	124.0	125.0	1.0
B	1,030	50	170	3.9	126.0	126.0	126.6	0.6
C	1,105	50	250	1.8	126.4	126.4	127.0	0.6
D	1,190	50	230	2.0	126.5	126.5	127.1	0.6
E	1,240	50	400	1.1	126.5	126.5	127.1	0.6
F	3,185	50	300	1.1	126.6	126.6	127.3	0.7
G	4,025	50	280	0.7	126.6	126.6	127.3	0.7
Unnamed Brook								
H	4,075	50	210	0.6	126.6	126.6	127.3	0.7
I	4,750	50	95	1.3	127.0	127.0	127.7	0.7
J	4,965	50	170	0.7	127.1	127.1	127.8	0.7
K	5,755	50	95	0.6	127.1	127.1	127.9	0.8

‘Feet above Rockingham park culvert

TABLE 11

ROCKINGHAM COUNTY, NH

(ALL JURISDICTIONS)

FLOODWAY DATA

POLICY BROOK – UNNAMED BROOK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NGVD29)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Shields Brook								
A	1.149	20	45	8.2	263.8	263.8	263.8	0.0
B	1.415	16	96	3.8	276.3	276.3	276.3	0.0
C	1.815	45	47	5.9	294.0	294.0	294.0	0.0
D	1.949	30	41	6.7	297.9	297.9	297.9	0.0
E	2.030	47	158	1.7	301.6	301.6	302.2	0.6
F	2.116	18	157	1.8	307.1	307.1	307.1	0.0
G	2.170	40	240	1.2	307.3	307.3	307.3	0.0
H	2.669	94	167	1.7	307.7	307.7	308.6	0.9
I	2.852	20	92	3.0	313.1	313.1	314.1	1.0
J	3.008	8	27	10.2	333.6	333.6	333.6	0.0
K	3.178	9	86	1.7	351.6	351.6	352.0	0.4
L	3.372	20	123	1.2	352.7	352.7	353.3	0.6
M	3.953	20	82	1.8	366.0	366.0	366.9	0.9
N	4.488	16	96	1.6	374.2	374.2	374.2	0.0

‘Miles above confluence with Beaver Creek

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH

(ALL JURISDICTIONS)

FLOODWAY DATA

SHIELDS BROOK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NGVD29)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Spicket River								
A	33.12	300	1,710	1.1	112.0	112.0	113.0	1.0
B	33.78	300	1,440	1.1	112.3	112.3	113.3	1.0
C	34.60	250	1,310	1.2	113.0	113.0	113.9	0.9
D	34.74	140	630	2.5	114.4	114.4	115.3	0.9
E	35.05	250	1,680	1.0	114.9	114.9	115.7	0.8
F	35.62	250	1,560	1.0	115.0	115.0	115.8	0.8
G	36.45	250	1,420	1.1	115.5	115.5	116.2	0.7
H	36.92	190	1,180	1.4	115.7	115.7	116.4	0.7
I	36.97	300	1,500	1.1	116.5	116.5	117.2	0.7
J	38.05	300	2,040	0.8	117.3	117.3	118.0	0.7
K	38.46	300	980	1.6	117.5	117.5	118.2	0.7
L	38.93	100	620	2.6	119.0	119.0	119.3	0.3
M	38.98	100	560	2.9	119.6	119.6	119.7	0.1
N	39.27	200	1,320	1.2	119.7	119.7	120.2	0.5
0	39.59	130	730	2.2	119.8	119.8	120.3	0.5
P	39.64	250	1,340	1.2	119.9	119.9	120.4	0.5
Q	40.66	250	1,380	1.2	120.6	120.6	121.1	0.5
R	40.82	250	1,500	1.2	120.7	120.7	121.3	0.6
S	40.87	250	1,840	0.8	121.8	121.8	122.5	0.7
T	41.87	180	760	1.8	122.3	122.3	122.9	0.6
U	42.47	200	1,350	1.0	126.3	126.3	126.3	0.0
V	42.74	60	460	1.6	126.4	126.4	126.5	0.1
W	43.11	100	450	1.7	127.1	127.1	127.2	0.1

‘Miles above Newburyport Light

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH
(ALL JURISDICTIONS)

FLOODWAY DATA

SPICKET RIVER

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NGVD29)			
CROSS SECTION	DISTANCE	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Taylor Brook (including Ballard Pond)								
A	0.225[1]	30	110	3.9	207.0	207.0	207.8	0.8
B	0.933[1]	19	87	4.9	218.2	218.2	218.9	0.7
C	1.638[1]	20	58	7.3	238.5	238.5	238.9	0.4
D	2.950/	208	1,085	0.8	258.4	258.4	259.4	1.0
E	3.153[1]	49	553	1.5	262.9	262.9	262.9	0.0
Tributary C to Beaver Brook								
A	0.092[2]	70	290	1.3	223.4	219.4[3]	220.3	0.9
B	0.571[2]	25	52	7.3	234.3	234.3	234.3	0.0
C	0.755[2]	30	51	7.5	247.1	247.1	247.1	0.0
D	0.960[2]	20	187	1.3	279.0	279.0	279.0	0.0
E	1.310[2]	40	47	5.1	292.3	292.3	292.3	0.0
F	1.800[2]	80	202	1.2	299.6	299.6	300.1	0.5
G	2.215[2]	160	230	1.0	304.6	304.6	305.6	1.0
Tributary G to Beaver Brook								
A	0.395[2]	50	489	1.5	248.0	243.7[3]	244.7	1.0
B	0.822[2]	18	532	1.0	265.4	265.4	265.8	0.4
C	1.181[2]	81	547	0.9	273.2	273.2	274.0	0.8
D	1.735[2]	16	567	0.9	281.9	281.9	282.8	0.9

[1]Miles above confluence with Island Pond
[2]Miles above confluence with Beaver Brook
[3]Elevation computed without consideration of backwater effects from Beaver Brook

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH
(ALL JURISDICTIONS)

FLOODWAY DATA

TAYLOR BROOK (INCLUDING BALLARD POND) - TRIBUTARY C TO BEAVER BROOK – TRIBUTARY G TO BEAVER BROOK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NGVD29)			
CROSS SECTION	DISTANCE	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Tributary O to Beaver Brook								
A	0.019[1]	30	48	5.2	239.1	235.0[3]	235.3	0.3
B	0.184[1]	35	104	2.4	239.1	237.9[3]	238.7	0.8
C	0.387[1]	20	38	6.1	245.9	245.9	246.2	0.3
D	0.585[1]	20	107	2.2	283.6	283.6	283.6	0.0
E	0.726[1]	350	2,576	0.1	285.4	285.4	285.4	0.0
F	0.926[1]	20	38	6.1	286.1	286.1	286.1	0.0
G	1.009[1]	30	114	2.0	290.4	290.4	291.2	0.8
H	1.121[1]	10	92	2.5	292.1	292.1	292.9	0.8
I	1.234[1]	20	101	2.3	305.4	305.4	305.4	0.0
J	1.453[1]	10	29	7.9	320.3	320.3	320.5	0.2
Tributary E to Beaver Lake								
A	0.000[2]	28	162	2.3	289.6	289.6	290.6	1.0
B	0.184[2]	36	467	0.8	293.6	293.6	294.3	0.7
Tributary F to Beaver Lake								
A	0.169[2]	102	589	1.1	297.6	297.6	298.6	1.0
B	0.471[2]	311	1,133	0.6	299.3	299.3	300.2	0.9
C	0.770[2]	59	226	2.9	303.5	303.5	304.5	1.0
D	1.064[2]	19	65	10.1	320.7	320.7	320.7	0.0

[1]Miles above confluence with Beaver Brook
[2]Miles above confluence with Beaver Lake
[3]Elevation computed without consideration of backwater effects from Beaver Brook

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH
(ALL JURISDICTIONS)

FLOODWAY DATA

TRIBUTARY O TO BEAVER BROOK – TRIBUTARY E TO BEAVER LAKE - TRIBUTARY F TO BEAVER LAKE

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NGVD29)			
CROSS SECTION	DISTANCE	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Tributary J to Black Brook								
A	0.191[1]		33	5.0	215.4	215.4	216.0	0.6
B	0.400[1]	20	94	1.8	221.1	221.1	221.5	0.4
C	0.613[1]	60	207	0.8	221.2	221.2	221.9	0.7
D	0.951[1]	30	103	1.6	221.8	221.8	222.8	1.0
E	1.145[1]	30	75	2.2	224.5	224.5	225.4	0.9
Tributary H to Drew Brook								
A	0.235[2]	26	52	4.8	216.9	216.9	217.3	0.4
B	0.503[2]	10	60	4.2	226.1	226.1	226.4	0.3
C	0.810[2]	14	30	8.4	245.1	245.1	245.3	0.2
D	1.030[2]	13	33	7.6	263.6	263.6	264.1	0.5
E	1.156[2]	17	40	6.3	277.3	277.3	277.6	0.3
Tributary E to Little Cohas Brook								
A	0.240[3]	60	205	2.1	264.1	262.4[4]	263.2	0.8
B	0.700[3]	40	118	2.8	264.1	262.5[4]	263.5	1.0
C	0.950[3]	30	107	3.1	266.1	266.1	266.1	0.0
D	1.083[3]	20	127	2.3	272.5	272.5	272.7	0.2
E	1.300[3]	100	538	0.5	276.9	276.9	277.3	0.4
F	1.535[3]	25	168	1.7	279.6	279.6	280.1	0.5
G	1.596[3]	10	63	4.6	281.3	281.3	281.3	0.0

[1]Miles above confluence with Black Brook
[2]Miles above confluence with Drew Brook
[3]Miles above confluence with Little Cohas Brook
[4]Elevation computed without consideration of backwater effects from Little Cohas Brook

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH
(ALL JURISDICTIONS)

FLOODWAY DATA

TRIBUTARY J TO BLACK BROOK – TRIBUTARY H TO DREW BROOK - TRIBUTARY E TO LITTLE COHAS BROOK

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NGVD29)			
CROSS SECTION	DISTANCE	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Tributary H to Nesenkeag Brook								
A	0.065[1]	30	69	5.4	185.0	185.0	185.0	0.0
B	0.350[1]	20	21	7.6	202.1	202.1	202.1	0.0
C	0.700[1]	20	23	7.0	232.3	232.3	232.3	0.0
D	1.151[1]	35	121	1.3	236.2	236.2	237.0	0.8
Upper Beaver Brook								
A	0.120[2]	20	38	5.7	314.3	314.3	314.3	0.0
B	0.300[2]	20	68	3.2	319.4	319.4	319.5	0.1
C	0.592[2]	20	45	4.8	331.6	331.6	331.6	0.0
D	0.900[2]	150	390	0.6	331.6	331.6	332.5	0.9
E	1.415[2]	300	824	0.3	331.7	331.7	332.7	1.0

[1]Miles above confluence with Nesenkeag Brook

[2]Miles above confluence with Shields Brook

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH

(ALL JURISDICTIONS)

FLOODWAY DATA

TRIBUTARY H TO NESENKEAG BROOK – UPPER BEAVER BROOK

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Winnicut River								
A	1,200	32	112	1.8	40.9	40.9	40.9	0.0
B	3,040	*	112	1.8	41.8	41.8	42.6	0.8
C	4,240	97	261	0.8	42.3	42.3	43.3	1.0
D	4,372	51	239	0.8	44.5	44.5	44.5	0.0
E	6,272	*	74	2.7	44.6	44.6	45.1	0.5
F	7,472	54	223	0.9	44.8	44.8	45.5	0.7
G	7,662	*	126	1.6	48.7	48.7	48.9	0.2
H	9,762	505	2,667	0.1	48.7	48.7	48.9	0.2
I	12,322	90	581	0.3	48.7	48.7	49.0	0.3
J	13,842	256	630	0.3	48.7	48.7	49.0	0.3
K	14,056	250	1,866	0.1	52.5	52.5	52.6	0.1
L	15,056	240	1,060	0.2	52.5	52.5	52.6	0.1
M	15,279	340	3,607	0.1	55.8	55.8	55.8	0.0

[1]Feet above Town of North Hampton corporate limits
*Floodway coincident with channel banks

TABLE 11

FEDERAL EMERGENCY MANAGEMENT AGENCY

ROCKINGHAM COUNTY, NH

(ALL JURISDICTIONS)

FLOODWAY DATA

WINNICUT RIVER

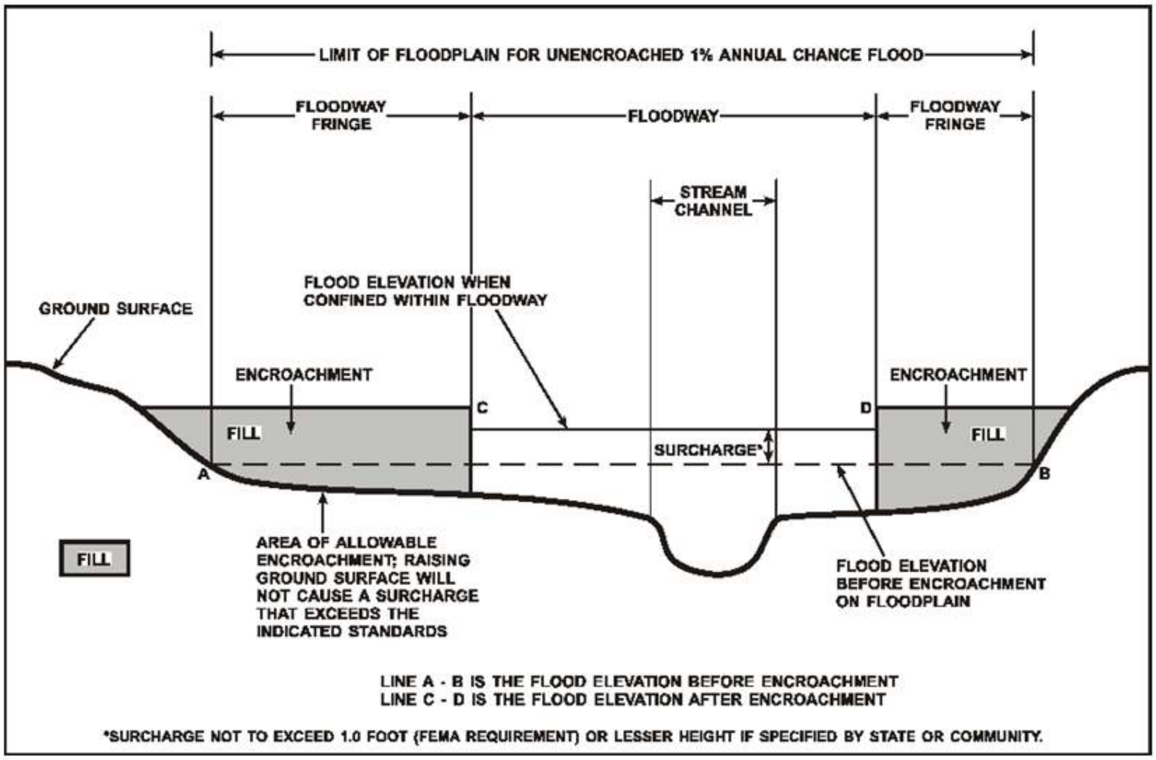
Portions of the floodways for Beaver Brook extend beyond the county boundary. No floodway was computed for Grassy Brook, Hill Brook, Hog Hill Brook, Porcupine Brook, Porcupine Brook Tributary, Powwow River (Downstream Reach), Powwow River (Upstream Reach), Squamscott River, Wash Pond Tributary, West Channel Policy Brook, and portions of the Lamprey River and Pickering Brook.

Encroachment into areas subject to inundation by floodwaters having hazardous velocities aggravates the risk of flood damage, and heightens potential flood hazards by further increasing velocities. A listing of stream velocities at selected cross sections is provided in Table 11, "Floodway Data." In order to reduce the risk of property damage in areas where the stream velocities are high, the community may wish to restrict development in areas outside the floodway.

Near the mouths of streams studied in detail, floodway computations are made without regard to flood elevations on the receiving water body. Therefore, "Without Floodway" elevations presented in Table 10 for certain downstream cross sections of Black Brook, Hidden Valley Brook, Homes Brook, Little River No. 1, Tributary C to Beaver Brook, Tributary G to Beaver Brook, Tributary O to Beaver Brook, Tributary E to Little Cohas Brook, and Tributary H to Nesenkeag Brook are lower than the regulatory flood elevations in that area, which must take into account the 1 percent annual chance flooding due to backwater from other sources.

The area between the floodway and 1 percent annual chance floodplain boundaries is termed the floodway fringe. The floodway fringe encompasses the portion of the floodplain that could be completely obstructed without increasing the water-surface elevation of the 1 percent annual chance flood by more than 1.0 foot at any point. Typical relationships between the floodway and the floodway fringe and their significance to floodplain development are shown in Figure 3.

Figure 3: Floodway Schematic



5.0 INSURANCE APPLICATIONS

For flood insurance rating purposes, flood insurance zone designations are assigned to a community based on the results of the engineering analyses. The zones are as follows:

Zone A

Zone A is the flood insurance rate zone that corresponds to the 1 percent annual chance floodplains that are determined in the FIS by approximate methods. Because detailed hydraulic analyses are not performed for such areas, no base flood elevations or depths are shown within this zone.

Zone AE

Zone AE is the flood insurance rate zone that corresponds to the 1 percent annual chance floodplains that are determined in the FIS by detailed methods. In most instances, whole-foot base flood elevations derived from the detailed hydraulic analyses are shown at selected intervals within this zone.

Zone AO

Zone AO is the flood insurance rate zone that corresponds to the areas of 1 percent annual chance shallow flooding (usually sheet flow on sloping terrain) where average depths are between 1 and 3 feet. Average whole-foot depths derived from the detailed hydraulic analyses are shown within this zone.

Zone V

Zone V is the flood insurance rate zone that corresponds to the 1 percent annual chance coastal floodplains that have additional hazards associated with storm waves. Because approximate hydraulic analyses are performed for such areas, no base flood elevations are shown within this zone.

Zone VE

Zone VE is the flood insurance rate zone that corresponds to the 1 percent annual chance coastal floodplains that have additional hazards associated with storm waves. Whole-foot base flood elevations derived from the detailed hydraulic analyses are shown at selected intervals within this zone.

Zone X

Zone X is the flood insurance rate zone that corresponds to areas outside of the 0.2 percent annual chance floodplain, areas within the 0.2 percent annual chance floodplain, and to areas of 1 percent annual chance flooding where average depths are less than 1 foot, areas of 1 percent annual chance flooding where the contributing drainage area is less than 1 square mile, and areas protected from the 1 percent annual chance flood by levees. No base flood elevations or depths are shown within this zone.

6.0 FLOOD INSURANCE RATE MAP

The FIRM is designed for flood insurance and floodplain management applications.

For flood insurance applications, the map designates flood insurance rate zones as described in Section 5.0 and, in the 1 percent annual chance floodplains that were studied by detailed methods, shows selected whole-foot base flood elevations or average depths. Insurance agents use the zones and base flood elevations in conjunction with information on structures and their contents to assign premium rates for flood insurance policies.

For floodplain management applications, the map shows by tints, screens, and symbols, the 1 percent annual chance and 0.2 percent annual chance floodplains. Floodways and the locations of selected cross sections used in the hydraulic analyses and floodway computations are shown where applicable.

The current FIRM presents flooding information for the entire geographic area of Rockingham County. Prior to the 2005 countywide study, separate FIRMs were prepared for each identified flood-prone incorporated community in the county. The countywide FIRM also included flood hazard information that was presented separately on FBFMs, where applicable. Historical data relating to the maps prepared for each community are presented in Table 12, "Community Map History."

TABLE 12 – COMMUNITY MAP HISTORY

Community Name	Initial Identification	Flood Hazard Boundary Map Revisions Date	FIRM Effective Date	FIRM Revisions Date
Atkinson, Town of	January 3, 1975	November 29, 1977	April 2, 1993	May 17, 2005
Auburn, Town of	February 28, 1975	None	April 2, 1986	May 17, 2005
Brentwood, Town of	June 28, 1974	December 10, 1976	April 15, 1981	May 4, 2000 May 17, 2005 January 29, 2021
Candia, Town of	June 28, 1974	November 19, 1976	May 17, 2005	None
Chester, Town of	February 21, 1975	None	March 1, 2000	May 17, 2005
Danville, Town of	January 17, 1975	None	April 1, 1994	May 17, 2005
Deerfield, Town of	June 28, 1974	November 12, 1976	September 1, 1989	May 17, 2005
Derry, Town of	September 13, 1974	March 4, 1977	April 15, 1981	May 17, 2005
East Kingston, Town of	February 28, 1975	None	April 2, 1986	May 17, 2005 January 29, 2021
Epping, Town of	July 19, 1974	November 15, 1977	April 15, 1982	May 17, 2005 January 29, 2021
Exeter, Town of	September 20, 1974	March 11, 1977	May 17, 1982	May 17, 2005 January 29, 2021
Fremont, Town of	August 9, 1974	October 29, 1976	April 15, 1981	June 19, 1989 May 17, 2005
Greenland, Town of	February 21, 1975	September 17, 1976	May 17, 1989	May 17, 2005 January 29, 2021
Hampstead, Town of	February 28, 1975	None	June 16, 1993	May 17, 2005
Hampton, Town of	July 19, 1974	December 10, 1976	July 3, 1986	May 17, 2005 January 29, 2021
Hampton Falls, Town of	December 6, 1974	June 11, 1976	April 15, 1982	May 17, 2005 January 29, 2021
Kensington, Town of	January 31, 1975	September 6, 1977	May 17, 2005	January 29, 2021
Kingston, Town of	January 17, 1975	March 6, 1979	September 1, 1988	April 15, 1992 May 17, 2005 January 29, 2021
Little Boar's Head, Village District of	February 27, 1979[2]	None	June 3, 1986[2]	May 17, 2005[2] January 29, 2021
Londonderry, Town of	August 9, 1974	July 16, 1976	November 5, 1980	May 17, 2005
New Castle, Town of	May 31, 1974	December 3, 1976	August 5, 1986	May 17, 2005 January 29, 2021
Newfields, Town of	January 17, 1975	March 12, 1976	June 5, 1989	May 17, 2005 January 29, 2021
Newington, Town of	February 21, 1975	None	May 17, 2005	January 29, 2021
Newmarket, Town of	June 28, 1974	December 10, 1976	May 2, 1991	May 17, 2005 January 29, 2021
Newton, Town of[3]	May 17, 2005[3]	None	May 17, 2005[3]	None

TABLE 12 – COMMUNITY MAP HISTORY - continued

Community Name	Initial Identification	Flood Hazard Boundary Map Revisions Date	FIRM Effective Date	FIRM Revisions Date
North Hampton, Town of	February 27, 1979	None	June 3, 1986	May 17, 2005 January 29, 2021
Northwood, Town of	January 2, 1987	None	January 2, 1987	May 17, 2005
Nottingham, Town of	June 28, 1974	November 19, 1976 September 7, 1979	April 2, 1986	May 17, 2005 January 29, 2021
Plaistow, Town of	October 18, 1974	August 27, 1976	April 15, 1981	May 17, 2005
Portsmouth, City of	July 19, 1974	July 23, 1976	May 17, 1982	May 17, 2005 January 29, 2021
Raymond, Town of	August 9, 1974	July 2, 1976	April 15, 1982	April 15, 1992 May 2, 1995 May 17, 2005
Rye, Town of	June 28, 1974	September 3, 1976	June 17, 1986	May 17, 2005 January 29, 2021
Salem, Town of	April 29, 1977	None	June 15, 1979	April 6, 1998 May 17, 2005
Sandown, Town of	January 3, 1975	None	January 1, 2003	May 17, 2005
Seabrook, Town of	August 2, 1974	November 26, 1976	June 17, 1986	May 17, 2005 January 29, 2021
Seabrook Beach Village District	August 2, 1974[1]	November 26, 1976[1]	August 5, 1986	May 17, 2005 January 29, 2021
South Hampton, Town of	February 28, 1975	None	June 1, 1989	July 15, 1992 May 17, 2005 January 29, 2021
Stratham, Town of	February 28, 1975	None	May 17, 1989	May 17, 2005 January 29, 2021
Windham, Town of	August 16, 1974	January 23, 1976	April 1, 1980	November 3, 1989 May 17, 2005

[1] The land area for this community was previously shown on the FHBM for the Town of Seabrook as a portion of the town. It has now been identified as a separate NFIP community. Therefore, the dates for this community were taken from the FHBM for the Town of Seabrook.
[2] The land area for this community was previously shown on the FIRM for the Town of North Hampton as a portion of the town. It has now been identified as a separate NFIP community. Therefore, the dates for this community were taken from the FIRM for the Town of North Hampton.
[3] This community did not have a published map prior to the first time countywide for Rockingham County, New Hampshire.

7.0 **OTHER STUDIES**

Information pertaining to revised and unrevised flood hazards for each jurisdiction within Rockingham County has been compiled into this FIS. Therefore, this FIS supersedes all previously printed FIS reports, FBFMs, and FIRMs for all jurisdictions within Rockingham County.

This is a multi-volume FIS. Each volume may be revised separately, in which case it supersedes the previously printed volume. Users should refer to the Table of Contents in Volume 1 for the current effective date of each volume; volumes bearing these dates contain the most up-to-date flood hazard data.

8.0 LOCATION OF DATA

Information concerning the pertinent data used in the preparation of this FIS Report can be obtained by submitting an order with any required payment to the FEMA Engineering Library. For more information on this process, see http://www.fema.gov.

9.0 BIBLIOGRAPHY AND REFERENCES

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Appendix A

Figure 4: FIRM Notes to Users

NOTES TO USERS

For information and questions about this map, available products associated with this FIRM including historic versions of this FIRM, how to order products, or the National Flood Insurance Program in general, please call the FEMA Mapping and Insurance eXchange at 1-877-FEMA-MAP (1-877-336-2627) or visit the FEMA Map Service Center website at http://msc.fema.gov. Available products may include previously issued Letters of Map Change, a Flood Insurance Study Report, and/or digital versions of this map. Many of these products can be ordered or obtained directly from the website. Users may determine the current map date for each FIRM panel by visiting the FEMA Map Service Center website or by calling the FEMA Mapping and Insurance eXchange.

Communities annexing land on adjacent FIRM panels must obtain a current copy of the adjacent panel as well as the current FIRM Index. These may be ordered directly from the Map Service Center at the number listed above.

For community and countywide map dates, refer to Section 6 this FIS Report.

To determine if flood insurance is available in the community, contact your insurance agent or call the National Flood Insurance Program at 1-800-638-6620.

The map is for use in administering the NFIP. It may not identify all areas subject to flooding, particularly from local drainage sources of small size. Consult the community map repository to find updated or additional flood hazard information.

BASE FLOOD ELEVATIONS: For more detailed information in areas where Base Flood Elevations (BFEs) and/or floodways have been determined, consult the Flood Profiles and Floodway Data and/or Summary of Stillwater Elevations tables within this FIS Report. Use the flood elevation data within the FIS Report in conjunction with the FIRM for construction and/or floodplain management.

Coastal Base Flood Elevations shown on the map apply only landward of 0.0' North American Vertical Datum of 1988 (NAVD88). Coastal flood elevations are also provided in the Coastal Transect Parameters table in the FIS Report for this jurisdiction. Elevations shown in the Coastal Transect Parameters table should be used for construction and/or floodplain management purposes when they are higher than the elevations shown on the FIRM.

FLOODWAY INFORMATION: Boundaries of the floodways were computed at cross sections and interpolated between cross sections. The floodways were based on hydraulic considerations with regard to requirements of the National Flood Insurance Program. Floodway widths and other pertinent floodway data are provided in the FIS Report for this jurisdiction.

NOTES FOR FIRM INDEX

REVISIONS TO INDEX: As new studies are performed and FIRM panels are updated within Rockingham County, New Hampshire (All Jurisdictions), corresponding revisions to the FIRM Index will be incorporated within the FIS Report to reflect the effective dates of those panels. Please refer to Table 12 of this FIS Report to determine the most recent FIRM revision date for each community. The most recent FIRM panel effective date will correspond to the most recent index date.

SPECIAL NOTES FOR SPECIFIC FIRM PANELS

This Notes to Users section was created specifically for Rockingham County, New Hampshire (All Jurisdictions), effective January 29, 2021.

LIMIT OF MODERATE WAVE ACTION: The Zone AE category has been divided by a Limit of Moderate Wave Action (LiMWA). The LiMWA represents the approximate landward limit of the 1.5-foot breaking wave. The effects of wave hazards between the VE Zone and the LiMWA (or between the shoreline and the LiMWA for areas where VE Zones are not identified) will be similar to, but less severe than those in the VE Zone.

Flood elevations on this map are referenced to either the National Geodetic Vertical Datum of 1929 (NGVD29) or the North American Vertical Datum of 1988 (NAVD88). Additional information is available in Section 3 of the accompanying Flood Insurance Study report. Note that flood elevations must be compared to structure and ground elevations referenced to the same vertical datum. For information regarding conversion between the National Geodetic Vertical Datum of 1929 (NGVD29) and the North American Vertical Datum of 1988 (NAVD88), visit the National Geodetic Survey website at http://www.ngs.noaa.gov or contact the National Geodetic Survey at the following address:

NGS Information Services
NOAA, N/NGS12
National Geodetic Survey
SSMC-3, #9202
1315 East-West Highway
Silver Spring, Maryland 20910-3282
(301) 713-3242

FLOOD RISK REPORT: A Flood Risk Report (FRR) may be available for many of the flooding sources and communities referenced in this FIS Report. The FRR is provided to increase public awareness of flood risk by helping communities identify the areas within their jurisdictions that have the greatest risks. Although non-regulatory, the information provided within the FRR can assist communities in assessing and evaluating mitigation opportunities to reduce these risks. It can also be used by communities developing or updating flood risk mitigation plans. These plans allow communities to identify and evaluate opportunities to reduce potential loss of life and property. However, the FRR is not intended to be the final authoritative source of all flood risk data for a project area; rather, it should be used with other data sources to paint a comprehensive picture of flood risk.

Each FIRM panel contains an abbreviated legend for the features shown on the maps. However, the FIRM panel does not contain enough space to show the legend for all map features. Figure 5 shows the full legend of all map features. Note that not all of these features may appear on the FIRM panels in Rockingham County.

Figure 5: Map Legend for FIRM

SPECIAL FLOOD HAZARD AREAS: *The 1% annual chance flood, also known as the base flood or 100-year flood, has a 1% chance of happening or being exceeded each year. Special Flood Hazard Areas are subject to flooding by the 1% annual chance flood. The Base Flood Elevation is the water surface elevation of the 1% annual chance flood. The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights. See note for specific types. If the floodway is too narrow to be shown, a note is shown.*

	Special Flood Hazard Areas subject to inundation by the 1% annual chance flood (Zones A, AE, AH, AO, AR, A99, V and VE).
Zone A	The flood insurance rate zone that corresponds to the 1% annual chance floodplains. No base (1% annual chance) flood elevations (BFEs) or depths are shown within this zone.
Zone AE	The flood insurance rate zone that corresponds to the 1% annual chance floodplains. Base flood elevations derived from the hydraulic analyses are shown within this zone, either at cross section locations or as static whole-foot elevations that apply throughout the zone.
Zone AH	The flood insurance rate zone that corresponds to the areas of 1% annual chance shallow flooding (usually areas of ponding) where average depths are between 1 and 3 feet. Whole-foot BFEs derived from the hydraulic analyses are shown at selected intervals within this zone.
Zone AO	The flood insurance rate zone that corresponds to the areas of 1% annual chance shallow flooding (usually sheet flow on sloping terrain) where average depths are between 1 and 3 feet. Average whole-foot depths derived from the hydraulic analyses are shown within this zone.
Zone AR	The flood insurance rate zone that corresponds to areas that were formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
Zone A99	The flood insurance rate zone that corresponds to areas of the 1% annual chance floodplain that will be protected by a Federal flood protection system where construction has reached specified statutory milestones. No base flood elevations or flood depths are shown within this zone.
Zone V	The flood insurance rate zone that corresponds to the 1% annual chance coastal floodplains that have additional hazards associated with storm waves. Base flood elevations are not shown within this zone.
Zone VE	Zone VE is the flood insurance rate zone that corresponds to the 1% annual chance coastal floodplains that have additional hazards associated with storm waves. Base flood elevations derived from the coastal analyses are shown within this zone as static whole-foot elevations that apply throughout the zone.
	Regulatory Floodway determined in Zone AE.
	Non-encroachment zone (see Section 2.4 of this FIS Report for more information)

OTHER AREAS OF FLOOD HAZARD

	Shaded Zone X: Areas of 0.2% annual chance flood hazards and areas of 1% annual chance flood hazards with average depths of less than 1 foot or with drainage areas less than 1 square mile.
	Future Conditions 1% Annual Chance Flood Hazard – Zone X: The flood insurance rate zone that corresponds to the 1% annual chance floodplains that are determined based on future-conditions hydrology. No base flood elevations or flood depths are shown within this zone.
	Area with Reduced Flood Risk due to Levee: Areas where an accredited levee, dike, or other flood control structure has reduced the flood risk from the 1% annual chance flood.

OTHER AREAS

	Zone D (Areas of Undetermined Flood Hazard): The flood insurance rate zone that corresponds to unstudied areas where flood hazards are undetermined, but possible.
NO SCREEN	Unshaded Zone X: Areas of minimal flood hazard.

FLOOD HAZARD AND OTHER BOUNDARY LINES

(ortho) (vector)	Flood Zone Boundary (white line on ortho-photography-based mapping; gray line on vector-based mapping)
	Limit of Study
	Jurisdiction Boundary
	Limit of Moderate Wave Action (LiMWA): Indicates the inland limit of the area affected by waves greater than 1.5 feet

GENERAL STRUCTURES

Aqueduct *Channel* *Culvert* *Storm Sewer*	Aqueduct, Channel, Culvert, or Storm Sewer
Dam *Jetty* *Weir*	Dam, Jetty, Weir
	Levee, Dike or Floodwall
Bridge	Bridge

REFERENCE MARKERS	
22.0	River Mile Markers
CROSS SECTION & TRANSECT INFORMATION	
B 20.2	Lettered Cross Section with Regulatory Water Surface Elevation (BFE)
5280 21.1	Numbered Cross Section with Regulatory Water Surface Elevation (BFE)
17.5	Unlettered Cross Section with Regulatory Water Surface Elevation (BFE)
8	Coastal Transect
	Profile Baseline: Indicates the modeled flow path of a stream and is shown on FIRM panels for all valid studies with profiles or otherwise established base flood elevation.
	Coastal Transect Baseline: Used in the coastal flood hazard model to represent the 0.0-foot elevation contour and the starting point for the transect and the measuring point for the coastal mapping.
513	Base Flood Elevation Line (shown for flooding sources for which no cross sections or profile are available)
ZONE AE (EL 16)	Static Base Flood Elevation value (shown under zone label)
ZONE AO (DEPTH 2)	Zone Designation with Depth
ZONE AO (DEPTH 2) (VEL 15 FPS)	Zone Designation with Depth and Velocity
BASE MAP FEATURES	
Lamprey River	River, Stream or Other Hydrographic Feature
234	Interstate Highway
234	U.S. Highway
234	State Highway
234	County Highway
MAPLE LANE	Street, Road, Avenue Name, or Private Drive if shown on Flood Profile
RAILROAD	Railroad
	Horizontal Reference Grid Line

—	Horizontal Reference Grid Ticks
+	Secondary Grid Crosshairs
Land Grant	Name of Land Grant
7	Section Number
R. 43 W. T. 22 N.	Range, Township Number
$^{42}76^{000m}E$	Horizontal Reference Grid Coordinates (UTM)
365000 FT	Horizontal Reference Grid Coordinates (State Plane)
80° 16' 52.5"	Corner Coordinates (Latitude, Longitude)

Table 13 is a list of the locations where FIRMs for Rockingham County can be viewed. Please note that the maps at these locations are for reference only and are not for distribution. Also, please note that only the maps for the community listed in the table are available at that particular repository. A user may need to visit another repository to view maps from an adjacent community.

TABLE 13 – MAP REPOSITORIES

Community	Address	City	State	Zip Code
Town of Atkinson	Town Office 21 Academy Avenue	Atkinson	New Hampshire	03811
Town of Auburn	Town Office 47 Chester Road	Auburn	New Hampshire	03032
Town of Brentwood	Town Hall 1 Dalton Road	Brentwood	New Hampshire	03833
Town of Candia	Town Office 74 High Street	Candia	New Hampshire	03034
Town of Chester	Municipal Office Building 84 Chester Street	Chester	New Hampshire	03036
Town of Danville	Town Office 210 Main Street	Danville	New Hampshire	03819
Town of Deerfield	Town Office 8 Raymond Road	Deerfield	New Hampshire	03037
Town of Derry	Derry Municipal Center 14 Manning Street	Derry	New Hampshire	03038
Town of East Kingston	Town Office 24 Depot Road	East Kingston	New Hampshire	03827

TABLE 13 – MAP REPOSITORIES - continued

Community	Address	City	State	Zip Code
Town of Epping	Town Hall 157 Main Street	Epping	New Hampshire	03042
Town of Exeter	Town Office 10 Front Street	Exeter	New Hampshire	03833
Town of Fremont	Town Hall 295 Main Street	Fremont	New Hampshire	03044
Town of Greenland	Town Office 11 Town Square	Greenland	New Hampshire	03840
Town of Hampstead	Town Hall 11 Main Street	Hampstead	New Hampshire	03841
Town of Hampton	Town Office 100 Winnacunnet Road	Hampton	New Hampshire	03842
Town of Hampton Falls	Town Hall 1 Drinkwater Road	Hampton Falls	New Hampshire	03844
Town of Kensington	Town Hall 95 Amesbury Road	Kensington	New Hampshire	03833
Town of Kingston	Town Office 163 Main Street	Kingston	New Hampshire	03848
Village District of Little Boar’s Head	North Hampton Town Office 233 Atlantic Avenue	North Hampton	New Hampshire	03862
Town of Londonderry	Town Office 268B Mammoth Road	Londonderry	New Hampshire	03053
Town of New Castle	Town Office 49 Main Street	New Castle	New Hampshire	03854
Town of Newfields	Town Hall 65 Main Street	Newfields	New Hampshire	03856
Town of Newington	Town Office 205 Nimble Hill Road	Newington	New Hampshire	03801
Town of Newmarket	Town Hall 186 Main Street	Newmarket	New Hampshire	03857
Town of Newton	Town Hall 2 Town Hall Road	Newton	New Hampshire	03858
Town of North Hampton	Town Office 233 Atlantic Avenue 2nd Floor	North Hampton	New Hampshire	03862

<u>TABLE 13 – MAP REPOSITORIES</u> - continued

Community	Address	City	State	Zip Code
Town of Northwood	Town Hall 818 First New Hampshire Turnpike	Northwood	New Hampshire	03261
Town of Nottingham	Town Hall 139 Stage Road	Nottingham	New Hampshire	03290
Town of Plaistow	Town Office 145 Main Street	Plaistow	New Hampshire	03865
City of Portsmouth	City Hall 1 Junkins Avenue	Portsmouth	New Hampshire	03801
Town of Raymond	Town Office 4 Epping Street	Raymond	New Hampshire	03077
Town of Rye	Town Office 10 Central Road	Rye	New Hampshire	03870
Town of Salem	Town Office 33 Geremonty Drive	Salem	New Hampshire	03079
Town of Sandown	Town Office 320 Main Street	Sandown	New Hampshire	03873
Town of Seabrook	Town Office 99 Lafayette Road	Seabrook	New Hampshire	03874
Seabrook Beach Village District	Warren H. West Memorial Building 210 Ocean Boulevard	Seabrook	New Hampshire	03874
Town of South Hampton	Town Office 3 Hilldale Avenue	South Hampton	New Hampshire	03827
Town of Stratham	Town Office 10 Bunker Hill Avenue	Stratham	New Hampshire	03885
Town of Windham	Windham Town Administrative Offices 4 North Lowell Road	Windham	New Hampshire	03087

Jurisdictions Included in the Flood Insurance Study Project

This FIS Report covers the entire geographic area of Rockingham County, New Hampshire.

The jurisdictions that are included in this project area, along with the Community Identification Number (CID) for each community and the 8-digit Hydrologic Unit Codes (HUC-8) sub-basins affecting each, are shown in Table 14. The Flood Insurance Rate Map (FIRM) panel numbers that affect each community are listed. If the flood hazard data for the community is not included in this FIS Report, the location of that data is identified.

The location of flood hazard data for participating communities in multiple jurisdictions is also indicated in the table.

Jurisdictions that have no identified SFHAs as of the effective date of this study are indicated in the table. Changed conditions in these communities (such as urbanization or annexation) or the availability of new scientific or technical data about flood hazards could make it necessary to determine SFHAs in these jurisdictions in the future.

TABLE 14 – LISTING OF NFIP JURISDICTIONS

Community	CID	HUC-8 Sub-Basin(s)	Located on FIRM Panel(s)	If Not Included, Location of Flood Hazard Data
Town of Atkinson	330175	01070006	33015C0552E, 33015C0554E, 33015C0556E, 33015C0558E, 33015C0560E, 33015C0570E, 33015C0576E, 33015C0578E	
Town of Auburn	330176	01070006	33015C0145E, 33015C0165E, 33015C0170E, 33015C0307E, 33015C0309E, 33015C0328E, 33015C0330E, 33015C0335E, 33015C0337E, 33015C0341E	
Town of Brentwood	330125	01060003	33015C0215E, 33015C0218E, 33015C0220F, 33015C0379E, 33015C0380E, 33015C0381E, 33015C0382E, 33015C0383E, 33015C0384E	
Town of Candia	330126	01060003, 01070006	33015C0145E, 33015C0155E, 33015C0160E, 33015C0165E, 33015C0170E, 33015C0178E, 33015C0186E	
Town of Chester	330182	01060003, 01070006	33015C0170E, 33015C0335E, 33015C0341E, 33015C0342E, 33015C0355E, 33015C0360E, 33015C0365E	
Town of Danville	330199	01060003, 01070006	33015C0360E, 33015C0370E, 33015C0378E, 33015C0379E, 33015C0390E	

TABLE 14. LISTING OF NFIP JURISDICTIONS – continued

Community	CID	HUC-8 Sub-Basin(s)	Located on FIRM Panel(s)	If Not Included, Location of Flood Hazard Data
Town of Deerfield	330127	01060003, 01070006	33015C0060E, 33015C0065E, 33015C0070E, 33015C0090E, 33015C0095E, 33015C0155E, 33015C0160E, 33015C0178E, 33015C0180E, 33015C0185E	
Town of Derry	330128	01060003, 01070006	33015C0328E, 33015C0330E, 33015C0336E, 33015C0337E, 33015C0339E, 33015C0341E, 33015C0342E, 33015C0343E, 33015C0344E, 33015C0363E, 33015C0365E, 33015C0527E, 33015C0529E, 33015C0531E, 33015C0532E, 33015C0533E, 33015C0551E, 33015C0552E	
Town of East Kingston	330203	01060003, 01070006	33015C0383E, 33015C0384E, 33015C0395E, 33015C0403E, 33015C0413E, 33015C0415E	
Town of Epping	330129	01060003	33015C0185E, 33015C0192E, 33015C0194E, 33015C0205E, 33015C0210F, 33015C0215E, 33015C0218E, 33015C0220F	
Town of Exeter	330130	01060003	33015C0220F, 33015C0236F, 33015C0238F, 33015C0239F, 33015C0245F, 33015C0382E, 33015C0384E, 33015C0401E, 33015C0402E, 33015C0403E, 33015C0404E, 33015C0406E, 33015C0408E, 33015C0410F	
Town of Fremont	330131	01060003	33015C0193E, 33015C0194E, 33015C0215E, 33015C0360E, 33015C0378E, 33015C0379E, 33015C0380E	
Town of Greenland	330210	01060003	33015C0235F, 33015C0245F, 33015C0255F, 33015C0265F, 33015C0270F	
Town of Hampstead	330211	01060003, 01070006	33015C0363E, 33015C0365E, 33015C0370E, 33015C0390E, 33015C0552E, 33015C0556E, 33015C0560E	
Town of Hampton	330132	01060003	33015C0410F, 33015C0428F, 33015C0430F, 33015C0433F, 33015C0436F, 33015C0437F, 33015C0439F, 33015C0441F, 33015C0443F	
Town of Hampton Falls	330133	01060003	33015C0408E, 33015C0410F, 33015C0420F, 33015C0428F, 33015C0436F, 33015C0437F, 33015C0438F, 33015C0439F	
Town of Kensington	330216	01060003, 01070006	33015C0403E, 33015C0404E, 33015C0408E, 33015C0413E, 33015C0415E, 33015C0420F	
Town of Kingston	330217	01060003, 01070006	33015C0370E, 33015C0378E, 33015C0379E, 33015C0383E, 33015C0384E, 33015C0390E, 33015C0395E, 33015C0403E, 33015C0576E, 33015C0577E	

TABLE 14. LISTING OF NFIP JURISDICTIONS – continued

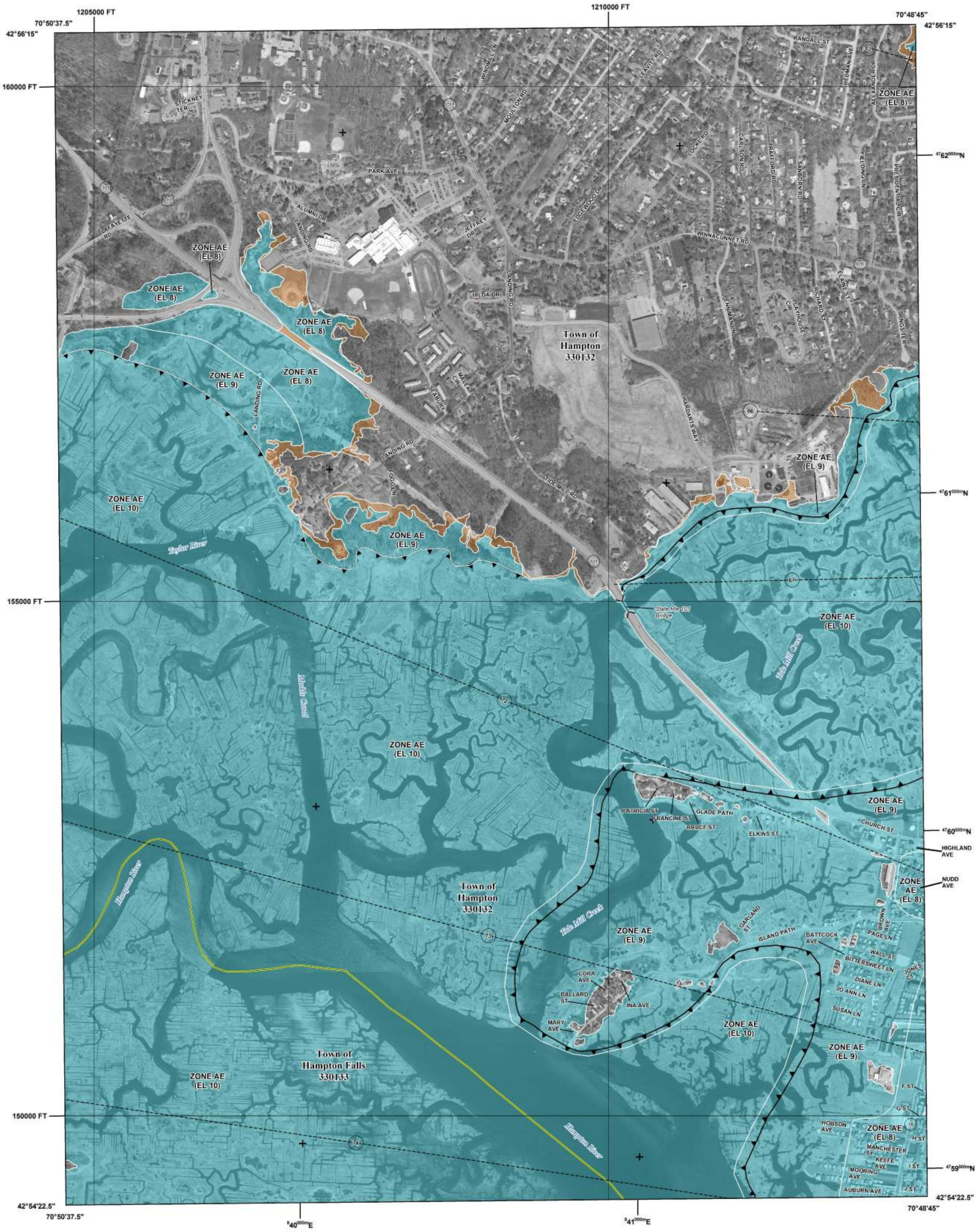
Community	CID	HUC-8 Sub-Basin(s)	Located on FIRM Panel(s)	If Not Included, Location of Flood Hazard Data
Village District of Little Boar's Head	330856	01060003	33015C0431F, 33015C0432F, 33015C0433F, 33015C0434F	
Town of Londonderry	330134	01070006	33015C0309E, 33015C0315E, 33015C0316E, 33015C0317E, 33015C0318E, 33015C0319E, 33015C0328E, 33015C0336E, 33015C0337E, 33015C0338E, 33015C0339E, 33015C0506E, 33015C0507E, 33015C0508E, 33015C0509E, 33015C0526E, 33015C0527E, 33015C0528E, 33015C0529E, 33015C0536E	
Town of New Castle	330135	01060003	33015C0278F, 33015C0279F, 33015C0286F, 33015C0287F	
Town of Newfields	330228	01060003	33015C0220F, 33015C0236F, 33015C0237F, 33015C0238F, 33015C0239F, 33015C0245F	
Town of Newington	330229	01060003	33015C0235F, 33015C0255F, 33015C0260F, 33015C0265F	
Town of Newmarket	330136	01060003	33015C0210F, 33015C0220F, 33015C0230F, 33015C0235F, 33015C0236F, 33015C0237F, 33015C0245F	
Town of Newton	330240	01070006	33015C0390E, 33015C0395E, 33015C0577E, 33015C0579E, 33015C0585E, 33015C0601E	
Town of North Hampton	330232	01060003	33015C0265F, 33015C0270F, 33015C0410F, 33015C0426F, 33015C0428F, 33015C0430F, 33015C0431F, 33015C0433F	
Town of Northwood	330855	01060003, 01070006	33015C0020E[1], 33015C0040E[1], 33015C0060E, 33015C0070E, 33015C0080E, 33015C0085E, 33015C0090E, 33015C0095E	
Town of Nottingham	330137	01060003	33015C0085E, 33015C0090E, 33015C0095E, 33015C0105E[1], 33015C0115E, 33015C0120E, 33015C0180E, 33015C0185E, 33015C0192E, 33015C0205E, 33015C0210F	
Town of Plaistow	330138	01070006	33015C0370E, 33015C0390E, 33015C0560E, 33015C0576E, 33015C0577E, 33015C0578E, 33015C0579E, 33015C0585E, 33015C0590E	
City of Portsmouth	330139	01060003	33015C0255F, 33015C0259F, 33015C0260F, 33015C0265F, 33015C0269F, 33015C0270F, 33015C0278F, 33015C0286F	

[1]Panel Not Printed

TABLE 14. LISTING OF NFIP JURISDICTIONS – continued

Community	CID	HUC-8 Sub-Basin(s)	Located on FIRM Panel(s)	If Not Included, Location of Flood Hazard Data
Town of Raymond	330140	01060003	33015C0170E, 33015C0178E, 33015C0180E, 33015C0185E, 33015C0186E, 33015C0187E, 33015C0190E, 33015C0191E, 33015C0192E, 33015C0193E, 33015C0194E, 33015C0335E, 33015C0355E, 33015C0360E	
Town of Rye	330141	01060003	33015C0265F, 33015C0269F, 33015C0270F, 33015C0286F, 33015C0287F, 33015C0288F, 33015C0431F, 33015C0432F, 33015C0434F, 33015C0451F, 33015C0457F, 33015C0459F, 33015C0476F, 33015C0478F	
Town of Salem	330142	01070006	33015C0543E, 33015C0545E, 33015C0551E, 33015C0552E, 33015C0553E, 33015C0554E, 33015C0558E, 33015C0561E, 33015C0562E, 33015C0563E, 33015C0564E, 33015C0570E, 33015C0657E[1], 33015C0676E, 33015C0677E, 33015C0681E	
Town of Sandown	330191	01060003, 01070006	33015C0355E, 33015C0360E, 33015C0365E, 33015C0370E	
Town of Seabrook	330143	01060003, 01070006	33015C0420F, 33015C0438F, 33015C0439F, 33015C0443F, 33015C0626F, 33015C0627F	
Seabrook Beach Village District	330854	01060003	33015C0439F, 33015C0627F	
Town of South Hampton	330193	01070006	33015C0395E, 33015C0413E, 33015C0415E, 33015C0420F, 33015C0585E, 33015C0601E, 33015C0602E	
Town of Stratham	330197	01060003	33015C0239F, 33015C0245F, 33015C0265F, 33015C0402E, 33015C0406E, 33015C0410F, 33015C0426F	
Town of Windham	330144	01070006	33015C0528E, 33015C0529E, 33015C0531E, 33015C0532E, 33015C0533E, 33015C0534E, 33015C0536E, 33015C0537E, 33015C0538E, 33015C0539E, 33015C0541E, 33015C0543E, 33015C0545E, 33015C0551E, 33015C0553E, 33015C0561E	

[1]Panel Not Printed



FLOOD HAZARD INFORMATION

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR FIRM PANEL LAYOUT

THE INFORMATION DEPICTED ON THIS MAP AND SUPPORTING DOCUMENTATION ARE ALSO AVAILABLE IN DIGITAL FORMAT AT

HTTPS://MSC.FEMA.GOV

SPECIAL FLOOD HAZARD AREAS	Without Base Flood Elevation (BFE) *Zone A, V, A99*
	With BFE or Depth *Zone AE, AO, AH, VE, AR*
	Regulatory Floodway
OTHER AREAS OF FLOOD HAZARD	0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
	Future Conditions 1% Annual Chance Flood Hazard *Zone X*
	Area with Reduced Flood Risk due to Levee See Notes. *Zone X*
	Area with Flood Risk due to Levee *Zone D*
OTHER AREAS	NO SCREEN Area of Minimal Flood Hazard *Zone X*
	Area of Undetermined Flood Hazard *Zone D*
GENERAL STRUCTURES	Channel, Culvert, or Storm Sewer
	Levee, Dike, or Floodwall
	E 18.2 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation
	8 Coastal Transect
	Coastal Transect Baseline
	Profile Baseline
	Hydrographic Feature
	513 Base Flood Elevation Line (BFE)
OTHER FEATURES	Limit of Study
	Jurisdiction Boundary

NOTES TO USERS

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Limit of Moderate Wave Action (LiMWA)

SCALE

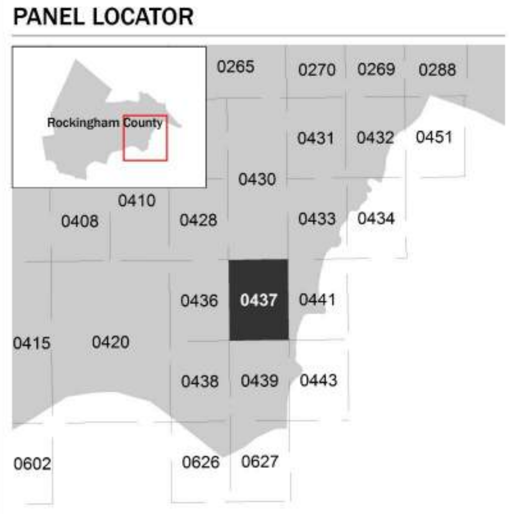
Map Projection:
NAD 1983 StatePlane New Hampshire FIPS 2800 Feet;
Western Hemisphere; Vertical Datum: NAVD 88

1 inch = 500 feet 1:6,000

0 500 1,000 2,000 Feet

0 130 260 520 Meters

PANEL LOCATOR



*PANEL NOT PRINTED

FEMA

National Flood Insurance Program

NATIONAL FLOOD INSURANCE PROGRAM

FLOOD INSURANCE RATE MAP

ROCKINGHAM COUNTY, NEW HAMPSHIRE
(All Jurisdictions)

PANEL **437** OF **681**

FEMA

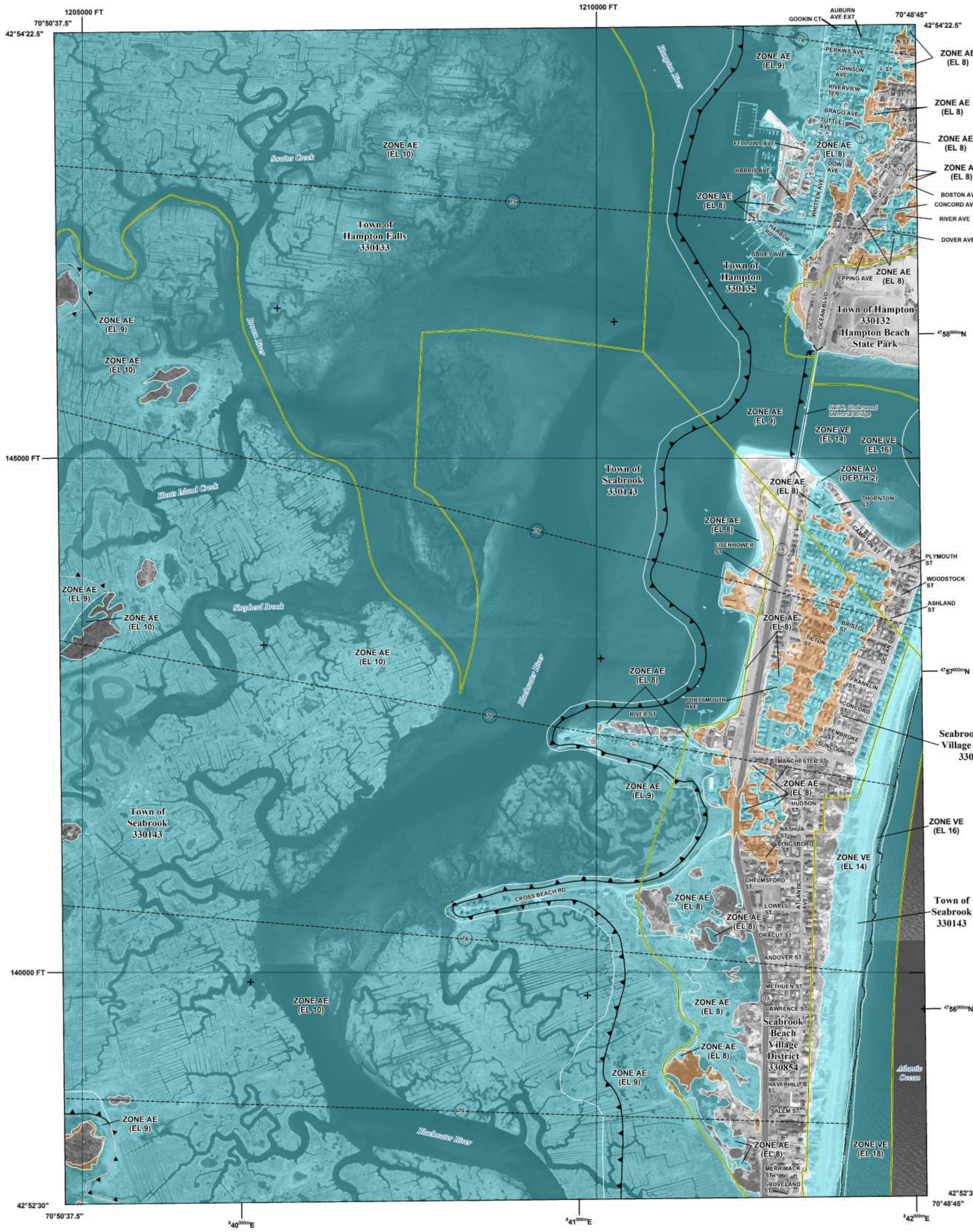
Panel Contains:

COMMUNITY	NUMBER	PANEL	SUFFIX
HAMPTON FALLS, TOWN OF	330133	0437	F
HAMPTON, TOWN OF	330132	0437	F

VERSION NUMBER
2.3.2.1

MAP NUMBER
33015C0437F

MAP REVISED
January 29, 2021



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	With BFE or Depth *Zone AE, AO, AH, VE, AR*
	Regulatory Floodway
OTHER AREAS OF FLOOD HAZARD	0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
	Future Conditions 1% Annual Chance Flood Hazard *Zone X*
	Area with Reduced Flood Risk due to Levee See Notes. *Zone X*
	Area with Flood Risk due to Levee *Zone D*
OTHER AREAS	NO SCREEN Area of Minimal Flood Hazard *Zone X*
	Area of Undetermined Flood Hazard *Zone D*
GENERAL STRUCTURES	Channel, Culvert, or Storm Sewer
	Levee, Dike, or Floodwall
OTHER FEATURES	E 18.2 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation
	8 Coastal Transect
	Coastal Transect Baseline
	Profile Baseline
	Hydrographic Feature
	513 Base Flood Elevation Line (BFE)
	Limit of Study
	Jurisdiction Boundary

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Limit of Moderate Wave Action (LiMWA)

SCALE

N

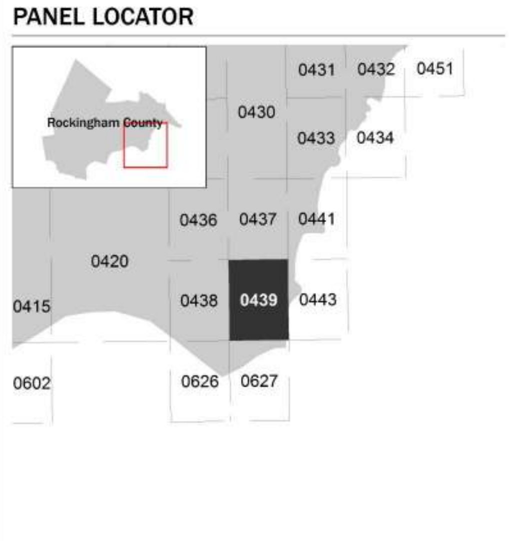
Map Projection:
NAD 1983 StatePlane New Hampshire FIPS 2800 Feet;
Western Hemisphere; Vertical Datum: NAVD 88

1 inch = 500 feet 1:6,000

0 500 1,000 2,000 Feet

0 130 260 520 Meters

PANEL LOCATOR



*PANEL NOT PRINTED

FEMA

National Flood Insurance Program

NATIONAL FLOOD INSURANCE PROGRAM

FLOOD INSURANCE RATE MAP

ROCKINGHAM COUNTY, NEW HAMPSHIRE
(All Jurisdictions)

PANEL **439** OF **681**

FEMA

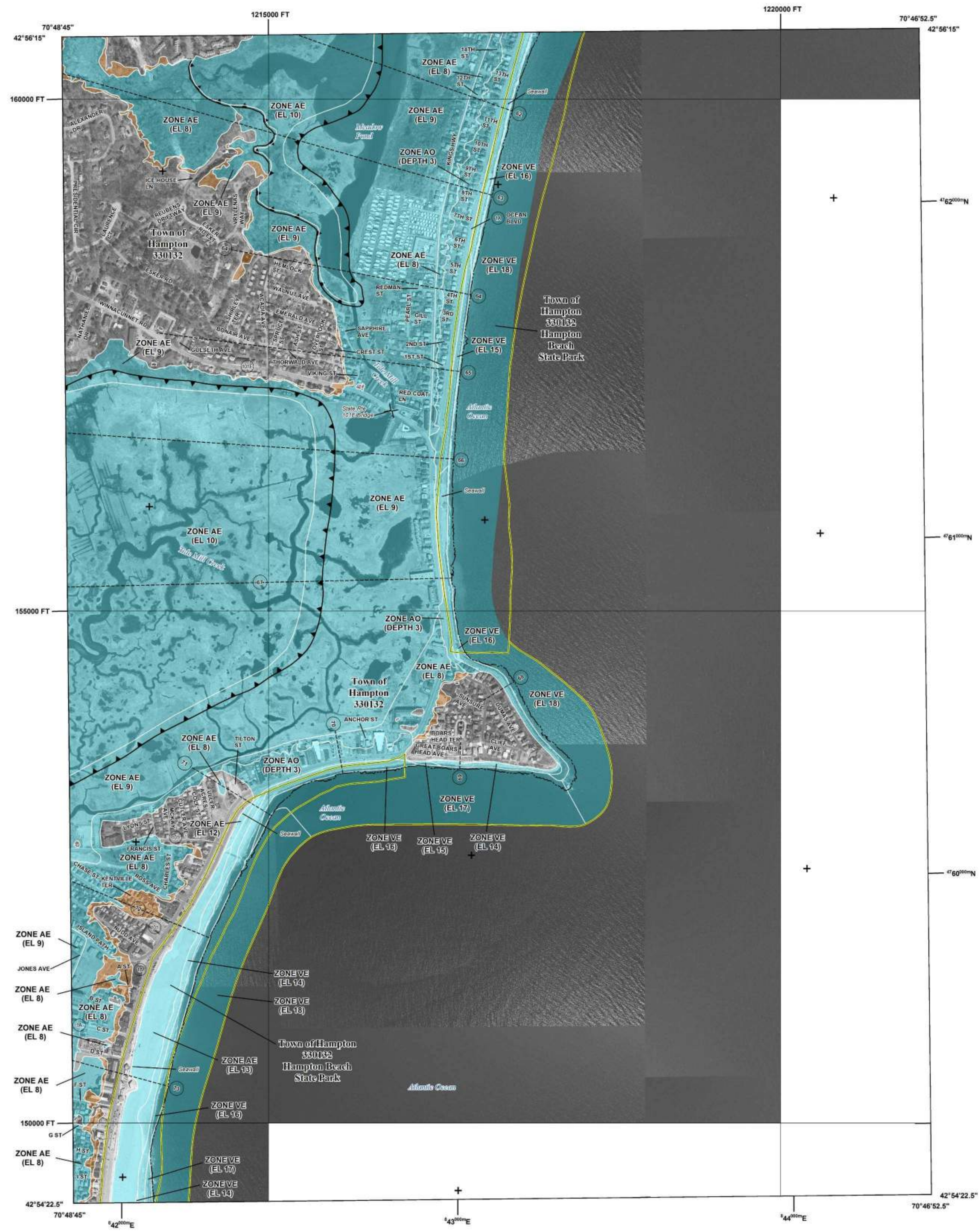
Panel Contains:

COMMUNITY	NUMBER	PANEL	SUFFIX
HAMPTON FALLS, TOWN OF	330133	0439	F
HAMPTON, TOWN OF	330132	0439	F
SEABROOK BEACH VILLAGE DISTRICT	330854	0439	F
SEABROOK, TOWN OF	330143	0439	F

VERSION NUMBER
2.3.2.1

MAP NUMBER
33015C0439F

MAP REVISED
January 29, 2021



FLOOD HAZARD INFORMATION

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		Regulatory Floodway
OTHER AREAS OF FLOOD HAZARD		0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
		Future Conditions 1% Annual Chance Flood Hazard *Zone X*
		Area with Reduced Flood Risk due to Levee See Notes. *Zone X*
		Area with Flood Risk due to Levee *Zone D*
OTHER AREAS	NO SCREEN	Area of Minimal Flood Hazard *Zone X*
		Area of Undetermined Flood Hazard *Zone D*
GENERAL STRUCTURES		Channel, Culvert, or Storm Sewer
		Levee, Dike, or Floodwall
	E 18.2 17.5	Cross Sections with 1% Annual Chance Water Surface Elevation
	8	Coastal Transect
		Coastal Transect Baseline
		Profile Baseline
		Hydrographic Feature
	513	Base Flood Elevation Line (BFE)
OTHER FEATURES		Limit of Study
		Jurisdiction Boundary

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Limit of Moderate Wave Action (LiMWA)

SCALE

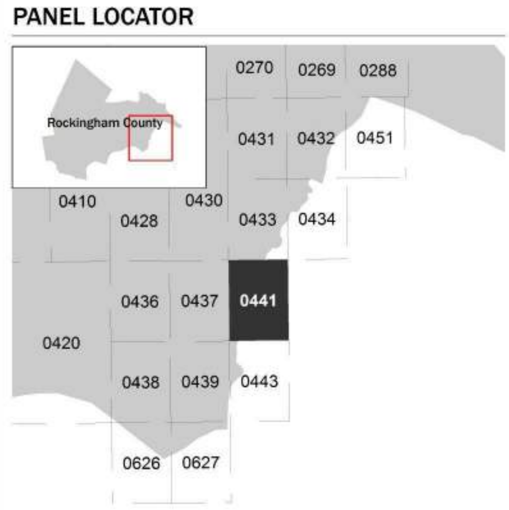
Map Projection:
NAD 1983 StatePlane New Hampshire FIPS 2800 Feet;
Western Hemisphere; Vertical Datum: NAVD 88

1 inch = 500 feet 1:6,000

0 500 1,000 2,000 Feet

0 130 260 520 Meters

PANEL LOCATOR



*PANEL NOT PRINTED

FEMA

National Flood Insurance Program

NATIONAL FLOOD INSURANCE PROGRAM

FLOOD INSURANCE RATE MAP

ROCKINGHAM COUNTY, NEW HAMPSHIRE
(All Jurisdictions)

PANEL **441** OF **681**

FEMA

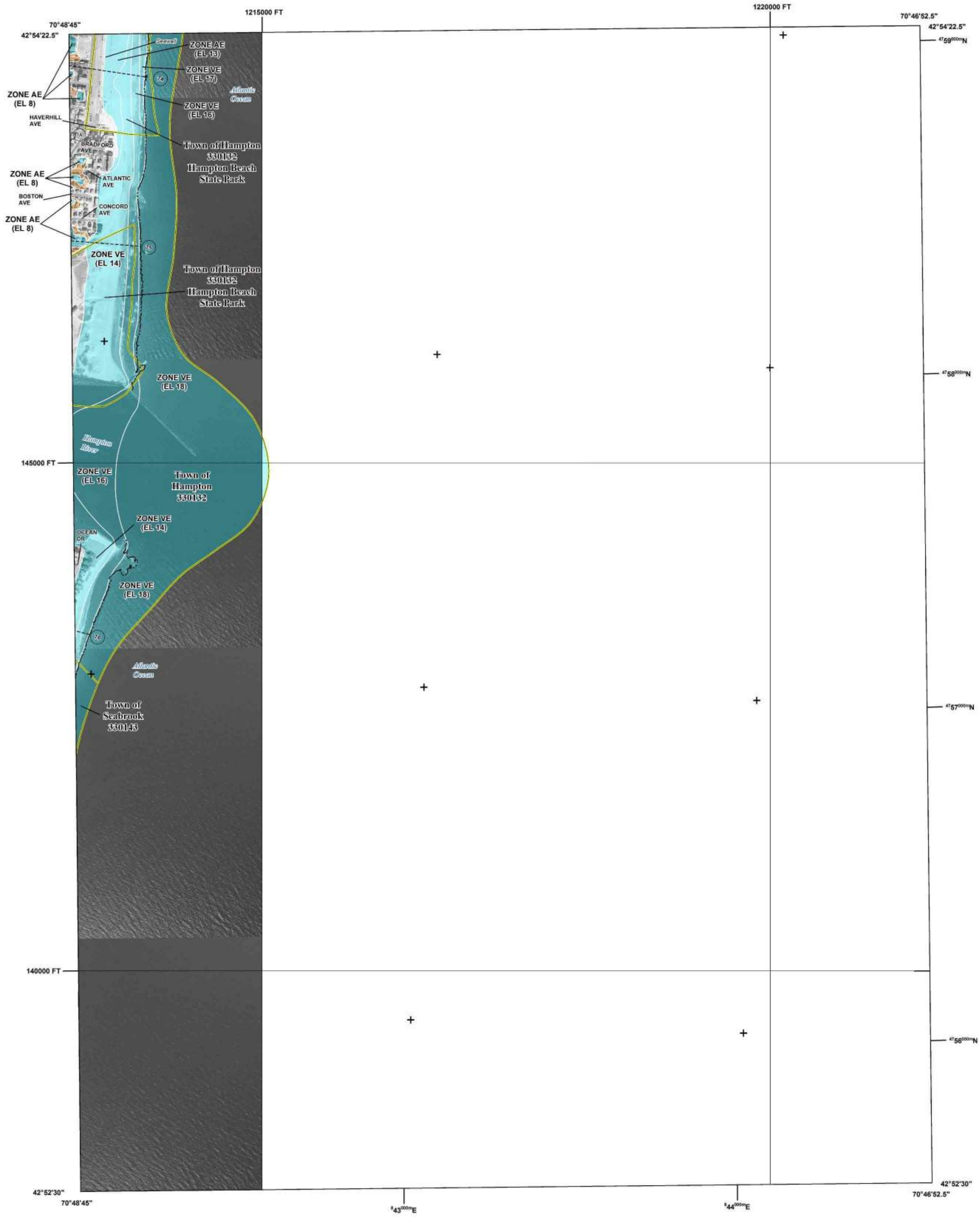
Panel Contains:

COMMUNITY	NUMBER	PANEL	SUFFIX
HAMPTON, TOWN OF	330132	0441	F

VERSION NUMBER
2.3.2.1

MAP NUMBER
33015C0441F

MAP REVISED
January 29, 2021



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	Regulatory Floodway
OTHER AREAS OF FLOOD HAZARD	0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
	Future Conditions 1% Annual Chance Flood Hazard *Zone X*
	Area with Reduced Flood Risk due to Levee See Notes. *Zone X*
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	Area of Undetermined Flood Hazard *Zone D*
GENERAL STRUCTURES	Channel, Culvert, or Storm Sewer
	Levee, Dike, or Floodwall
	E 18.2 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation
	8 Coastal Transect
	Coastal Transect Baseline
	Profile Baseline
	Hydrographic Feature
	513 Base Flood Elevation Line (BFE)
OTHER FEATURES	Limit of Study
	Jurisdiction Boundary

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SCALE

N

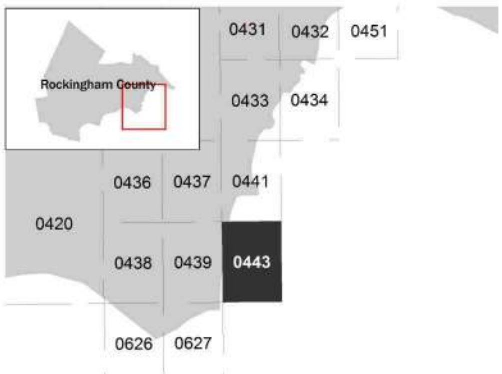
Map Projection:
NAD 1983 StatePlane New Hampshire FIPS 2800 Feet;
Western Hemisphere; Vertical Datum: NAVD 88

1 inch = 500 feet 1:6,000

0 500 1,000 2,000 Feet

0 130 260 520 Meters

PANEL LOCATOR



*PANEL NOT PRINTED

FEMA

National Flood Insurance Program

NATIONAL FLOOD INSURANCE PROGRAM

FLOOD INSURANCE RATE MAP

ROCKINGHAM COUNTY, NEW HAMPSHIRE
(All Jurisdictions)

PANEL **443** OF **681**

FEMA

Panel Contains:

COMMUNITY	NUMBER	PANEL	SUFFIX
HAMPTON, TOWN OF	330132	0443	F
SEABROOK, TOWN OF	330143	0443	F

VERSION NUMBER
2.3.2.1

MAP NUMBER
33015C0443F

MAP REVISED
January 29, 2021

APPENDIX H

ALTERNATIVES ANALYSIS EVALUATION MATRIX

Engineering Report

March 2021

Hampton Harbor Flood Mitigation Alternatives												
Category	Subcategory	Description	Objectives				Feasibility				Recommended to Further Investigate
			Reduce Flooding	Protect Private Property	Increase Public Safety	Minimize Environmental Impacts	Permitability	Constructability	Comparative Implementation Cost	Comparative Operation & Maintenance Cost	
Resist with Infrastructure	Permanent Barriers	Install walls at low areas of Glade Path	Good	Good	Fair	Fair	Poor	Good	Good	Good	X
		Install walls at low areas of Island Path	Good	Good	Fair	Fair	Poor	Fair	Fair	Good	X
		Install walls at low areas of Ashworth Ave	Good	Good	Fair	Poor	Poor	Poor	Poor	Fair	X
		Install walls at low areas of North of NH Route 101	Good	Good	Fair	Fair	Poor	Fair	Fair	Good	X
		Vegetated earthen berms	Good	Good	Fair	Poor	Poor	Poor	Fair	Good	X
	Temporary Barriers	Walls installed for duration of event and removed	Good	Fair	Good	Good	Good	Fair	Fair	Poor	X
	Elevate Flood-prone Roads	Glade Path	Fair	Poor	Good	Fair	Fair	Fair	Fair	Good	X
		Island Path	Fair	Poor	Fair	Fair	Fair	Fair	Fair	Good	X
		Ashworth Ave	Fair	Poor	Poor	Good	Good	Poor	Poor	Fair	
Accommodate with Infrastructure	Existing Infrastructure	Elevate flood-prone houses/buildings	Good	Fair	Fair	Good	Fair	Good	Fair	Good	X
		Zoning & site plan regulation changes	Good	Fair	Fair	Fair	Good	Good	Good	Good	X
		Improve drainage	Fair	Fair	Fair	Good	Good	Good	Good	Fair	
		Tide gate improvements on stormwater outfalls	Poor	Poor	Fair	Poor	Good	Good	Good	Fair	
Retreat	Property Changes	Voluntary and assisted retreat/relocation	Poor	Fair	Good	Good	Good	Good	Poor	Good	X

Hoyle, Tanner & Associates, Inc.